SRR-88-07

Load Testing and Evaluation of Turner's Bridge

Ontario

Ministry of Transportation

Research and Development Branch

Technical Report Documentation Page

Load Testing and Evaluation of Turner's Bridge

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Number:	SRR-88-07
Date of Publication:	November 1990
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Abstract:	Turner's bridge was evaluated in 1985 according to the OHBDC [1] specifications. This evaluation resulted in recommendations for considerable rehabilitation in order to avoid severe posting restrictions. The westerly truss underwent limited reinforcing of some critical diagonal members in 1985 as a means of keeping the bridge open. Due to the high costs associated with the remaining proposed reinforcing, the County of Bruce requested that the structure be load tested by MTO. The bridge was included in MTO's 1986 bridge testing program. The structure is comprised of two steel through truss spans, each supporting concrete decks on steel stringers and floor beams. Both truss spans were proof tested in July 1986 and the results of these tests and the subsequent evaluation are the subject of this report. The testing confirmed the use of frame analysis as a tool for evaluating the steel trusses. The report recommends a new triple posting of 18, 32 and 36 tonnes.
Comments:	
Key words:	bridge testing, steel truss,
Distribution:	MTO Head Office, key Ministry officials
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91-07205/

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April 23, 1991

Memorandum to: Distribution

Re: Structural Research Report SRR-88-07 "Load testing Evaluation of the Turners Bridge" MTO #2-066.

Please find attached a research report describing a load test on, and evaluation of, an old steel through truss bridge.

The preliminary analytical evaluation of the bridge led to a triple posting of 10, 15 and 25 tonnes. As a consequence of the test, this posting could be upgraded to 18, 31 and 33 tonnes.

The report contains all the relevant data recorded during the test.

Your comments on the report are welcome. Please address them to the author of this report.

for B. Bakht
Principal Research Engineer
Structures Office

RJT/ss

cc: R. Taylor

Load Testing and Evaluation of Turner's Bridge

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Structures Research Office
Research and Development Branch

Published by
The Research and Development Branch
Ontario Ministry of Transportation
Hon. E. Philip, Minister
Pat Jacobsen, Deputy Minister

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November 1990

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Test Summary

Bridge:	Turner's Bridge, Bruce County, MTO Site No. 2-066
Type of Structure:	Steel through trusses with concrete deck on steel stringers.
Posted Load:	10 tonnes
Span/Roadway Width:	48.8 m, 45.7 m / 4.5 m
Angle of Skew:	None
Date of Test:	July 29 and 31, 1986
Type of Test:	Proof
Maximum Static Load Applied:	Two vehicles @ 590 kN each. One vehicle @ 778 kN.
Responses Measured:	Strains at critical sections of truss members, stringers and floor beams

1/ Introduction

Turner's Bridge was evaluated in 1985 according to the Ontario Highway Bridge Design Code [1]. This evaluation resulted in recommendations for considerable rehabilitation in order to avoid severe posting restrictions. The westerly truss underwent limited reinforcing of some critical diagonal members in 1985 as a means of keeping the bridge open.

Due to the high costs associated with the remaining proposed reinforcement, the County of Bruce requested that the structure be load tested by the Ministry of Transportation of Ontario. The bridge was included in MTO's 1986 bridge testing program.

Both truss spans were proof tested in July 1986 and the results of these tests and the subsequent evaluation are the subject of this report. The report recommends a new triple posting of 18, 32 and 36 tonnes with no further reinforcement of either span.

2/ Site Description

Turner's Bridge, MTO Site No. 2-066, is a single-lane structure located on the 4th Concession of Saugeen Township in the County of Bruce. As shown in Figure 1, the two-span structure consists of two steel through trusses supporting steel stringers and floor beams with an exposed concrete deck. The westerly truss (Span No. 1) has a span of 48.77 m and the easterly truss (Span No. 2) spans 45.40 m. Dimensional and section size data are provided in Figures 2 through 4.

The steel members in the trusses are in reasonably good condition, as are most of the stringers and floor beams. Some secondary truss members of Span No. 1 have been strengthened by the addition of steel channels, as shown in Figure 2. Some of the original stringers at the west end of Span No. 1 were severely deteriorated and had been replaced and/or strengthened by new stringers, as shown in Figure 4.

3/ Preliminary Analysis and Evaluation

Where necessary, a preliminary analysis of the structures was performed to identify areas of critical stress under the influence of the proposed test loads. This analysis was also used as a basis for the derivation of the critical vehicle placement that was used during the actual testing.

3.1/ Deck System

No extensive analytical investigation was performed on the deck system prior to field testing. An evaluation of the stringers and floor beams using the OHBDC [1] simplified methods provided critical capacity reduction factors in Span No. 1 of F = 0.72 and F = 0.57, respectively. This assumed non-composition between the concrete deck and the supporting steel members. In Span No. 2 the critical capacity reduction factors were F = 0.78 and F = 0.87 for the stringers and floor beams, respectively.

In most cases, the steel stringers and floor beams of old bridges display partial composite action with the concrete deck. Therefore, there was little benefit in conducting any further analytical investigation prior to load testing. Considering the simple structural make-up of the deck arrangement, the maximum stress locations and the vehicle placements which caused them were determined using statics.

3.2/ Steel Trusses

The steel trusses of both spans were analysed as two-dimensional frames (using STRUDL [2]), as depicted in Figure 5. This analysis generated influence coefficients for all members under the influence of a single unit load placed at each panel point. The same analytical model was used to determine dead-load forces. This model considered moment connections at all joints, however, due to the slenderness of the members, only axial forces in the members were considered in the subsequent evaluation.

Using the results of the influence analysis in conjunction with an in-house program, which determines critical vehicle placement for given influence lines, the trusses were evaluated for the factored OHBDC design loads. Tables 1 and 2 provide the results of this evaluation in terms of the critical axial member forces due to factored OHBDC live loads and dead loads for Spans No. 1 and 2, respectively. These tables also include the factored member resistances determined according to

the OHBDC, assuming a material yield strength of F_y = 230 MPa and an effective length factor of K = 1.0 for compression.

The last columns in these tables provide the critical scale-down factor F for each member determined according to the OHBDC. These indicate that member 30 of Span No.1 is the governing case for all three evaluation levels. All other members provide scale-down factors of over 0.83. In Span No. 2 the critical member appears to be member 35 in compression, as it has a very low capacity.

In order to evaluate the effects of the potential buckling of member 35 in Span No.2, the frame analysis was re-run removing member 35 for loadings which would put this member in compression. Considering changes in both dead and live load responses, only member 29 displayed an increase in load effect exceeding more than 3 or 4 %. The bracketed values of critical scale-down factors for member 29, given in Table 2, are the result of ignoring member 35 in the analysis.

3.3/ Summary

Table 3 provides the calculated OHBDC scale-down factors for the trusses, stringers and floor beams for both spans. The floor beams in Span No. 1 govern for all three evaluation levels if the effects of member 35 in the truss of Span No. 2 are ignored. This would provide a required triple posting of 10, 15, and 25 tonnes. Further evaluation was to be conducted following the load testing.

4/ Instrumentation

The two spans were tested separately due to the limited number of instrumentation channels available for monitoring. In both cases all of the available 60 channels were utilized.

4.1/ Span No. 1

A total of 45 strain gauges were used to monitor steel strains in the truss members; 39 of these were on the north truss, as shown in Figure 6, and the other 6 were on the south truss, as shown in Figure 7. The locations marked with a "G" were temperature-compensated gauges which were to be used again for a long-term load survey. All other gauges were either weldable or demountable.

Only one bay of the deck system was instrumented, as shown in Figure 8. The third bay from the west end was selected in order to avoid the effects of the additional stringers that had been added in the first two bays. Thirteen strain gauges were applied to the floor members as shown in the figure; 10 on the stringers and 3 on the floor beams. Again, those locations marked with a "G" were to be used later for the vehicle survey.

The remaining two channels were used to monitor displacement transducers located at the west end of the span, as indicated in Figures 6 and 7. These transducers monitored horizontal movement of the span at the roller bearings with respect to the abutment.

Note that at Site No. 26, the gauge originally monitoring compressive strain in the top of the steel stringer was disconnected after load level 2 in order to monitor compressive strain in the top of floor beam No. 3 at Site No. 32.

4.2/ Span No. 2

Originally, a total of 42 strain gauges were used to monitor steel strains in the truss members; 33 of these were on the north truss, as shown in Figure 9, and nine were on the south truss, as shown in Figure 10. However, after load level 4, gauge No. 25 was disconnected (Figure 9) and this channel was used to monitor compression in the top chord of the south truss (Figure 10). In addition, since no other monitor channels were available and a number of additional gauge locations were requested on-site, a continuous readout strip-chart recorder was used after load level 4. This

chart recorder monitored two demountable strain gauges and two displacement transducers at Sites No. 12, 13 and 21, 22, respectively, as shown in Figure 9.

The last bay of the floor system at the east end of the span was instrumented along with floor beams No. 2 and 3 as shown in Figure 11. A total of 16 strain gauges were used; 12 on the stringers and 4 on the floor beams.

The remaining two channels were utilized as before, to monitor horizontal movement at the roller bearings with respect to the abutment (Figures 9 and 10).

4.3/ Equipment

The instrument gauges were monitored using MTO's 60-channel capacity data acquisition system which used a Motorola 6800 Computer Base. The demountable strain gauges have an accuracy of ±5 microstrains, considering the installation procedures. The electrical resistance strain gauges have an accuracy of about 1 or 2 microstrain(s). The displacement transducers have an accuracy of ±0.14 mm, considering the installation procedure.

5/ Load Testing

The load testing of the bridge was performed on July 29 and 31, 1986 for Spans No. 1 and 2, respectively. The two spans were tested separately due to limited instrumentation facilities.

5.1/ Vehicle Loads

The bridge was tested by using two MTO test vehicles, the dimensional configurations of which are given in Figure 12. The proposed test loads in terms of tandem/axle weights are provided in Tables 4 and 5.

5.2/ Vehicle Placement

The vehicles were placed so as to produce maximum responses at all critical sections on the bridges. The same line positions, or placement within the roadway width, were used on both structures, as depicted in Figure 13. Line 4 is the same as line 3 except that the second vehicle follows the first at a clear distance between axles of about 10 m.

The critical longitudinal positions, or positions along the bridge, were derived from the preliminary influence analysis model using the proposed test vehicle configurations for the maximum load levels. After identifying a number of critical positions, additional intermediate positions were then added to provide enough test data to be able to generate influence lines for the vehicles for each member. The final longitudinal positions used in the testing are given in Figures 14 and 15.

5.3/ Testing

Load testing was performed on July 29 and 31, 1986 for Spans No. 1 and 2, respectively. The maximum proposed test loads given in Tables 4 and 5 were successfully applied with no visual evidence of distress in either structure.

6/ Test Results

The following section provides the presentation and discussion of the test results including, where applicable, comparisons with analytical models.

6.1/ Steel Stringers

Figures 16 to 18 and 19 to 21 display some of the test results for the stringers in Spans No. 1 and 2, respectively. The plots give the steel strains in the top and/or bottom flanges of the stringers plotted on the same scale.

In both cases the strains indicate that some composite action is taking place between the concrete deck and steel stringers, with more composition evident in Span No. 2. However, the degree of composite action appears to vary, making it difficult to simulate the deck analytically for design purposes. Several of the maximum output gauges were checked against increasing load effects and were found to be linear within the accuracy of the instrumentation and vehicle placement.

6.1.1/ Span No. 1

Table 6 provides some of the tensile strains from the stringers in Span No. 1. If it is assumed that these strains represent the relative magnitude of the moment in each of the stringers, the critical distribution width for moment D_d for each of the loadings represented in Table 6 can be estimated. For any one vehicle position, the sum of the strains across the width of the deck divided into the maximum strain measured, results in the maximum fraction of the full vehicle effect carried by one stringer. The latter is given in column 4 of Table 7 for the data is in Table 6. These maximum load fractions represent one half of S/D_d which is accounted for in the determination of the distribution widths given in column 5 of Table 7, where the stringer spacing is S = 0.91 m.

The validity of this procedure is questionable as it assumes that all the stringer sections have the same stiffness. (This is probably not the case, due to the partial composite action between the slab and stringers. However, even considering some variation in section stiffness, these experimental distributions are considerably below the design value of 2.09 m derived from the OHBDC assuming non-composition between the slab and stringers. It should be noted that the values do not vary significantly throughout the increases in load and changes in vehicle positions. In light of the above, the actual distribution of moments in these stringers is probably worse than that

assumed in the design process and this should be taken into consideration when re-evaluating the bridge. Further discussion will be presented in Section 7.

If all the stringers had had strain gauges on the top flanges as well as on the bottom flanges it would have been possible to carry the investigation further. This was the case with the stringers in Span No. 2 as will be presented in the following section.

6.1.2/ Span No. 2

Tables 8 and 9 represent the same procedure for determining the distribution widths for the stringers in Span No. 2, as applied in Section 6.1.1. The same concerns apply with regard to the validity of the procedure. In addition the values are again quite consistent throughout the increases in load and changes in vehicle position. The magnitude of the distribution widths is larger in this deck than in Span No. 1. Here, the design distribution width is 2.15 m and the experimental values are very close to this.

Unlike the stringers in Span No. 1, these stringers had strain gauges on the top and bottom flanges as shown in Figures 19 to 21. The strain results are tabulated in Table 10 for load levels 3 and 5. Using these results, a simple procedure to determine the total moment across the deck at the instrumented location can be applied. The procedure, presented in detail by Bakht [3], utilizes measured strains in the top and bottom flanges of a partially or fully composite slab-on-girder bridge. The method breaks down the computation into three parts as follows:

$$M = M_L + M_U + N * a$$

Here M_L and M_U are the moments taken by the naked girder and the slab respectively, bending about their own axis. N is the axial force acting in opposite directions in the slab and girder to a lever arm, a. The components of this equation can be determined as follows:

$$M_L = \sigma_{fb} S_b$$
$$M_U = M_L (EI_s/EI_g)$$
$$Na = \sigma_a Aa$$

where σ_{fb} = flexural stress in the bottom flange of steel stringer

S_b = section modulus of the naked steel stringer with respect to bottom flange

EI_g = flexural rigidity of the naked steel stringer bending about its own neutral axis

EI_s = flexural rigidity of the slab bending about its own neutral axis

σ_a = axial stress in the naked steel stringer due to axial force N

A = cross-sectional area of naked steel stringer

a = lever arm of the couple due to axial forces N (assumed to be the distance between neutral axes of stringer and slab)

Using this method the section moments from the strain data in Table 10 are summarized in Table 11. In the last column a distribution width was determined based on the maximum calculated moment for each case. These distribution widths should represent a better approximation than those using only the tensile strains, as these take into account the partial composite action. However, these values compare very well with those given in Table 9, indicating that the method using the tensile strains is a reasonably good approximation for this deck.

Table 12 summarizes the total section moments from the experimental data as compared to the total predicted moments due to statics. Two values of predicted moment are given; the first represents the assumption that the stringer spans are simply supported at the floor beams, and the second assumes that the deck is continuous over the floor beams. The test results indicate that the actual performance is such that the deck system is close to being fully continuous over the floor beams. It should be noted that the total section moments from the experimental results as given in Table 11 or 12 are nearly constant in each load level, independent of the vehicles position across the width of the deck.

6.1.3/ Summary

It appears that both deck systems demonstrate some composite action between the slab and stringers. The calculations performed on Span No. 2 showed that the deck system was nearly fully continuous over the floor beams. The same performance could not be confirmed in Span No. 1. In the future, it is recommended that all stringers/girders of slab-on-girder bridges be instrumented top and bottom to allow the determination of moments across the section. This would provide a better record of the distribution performance as well as allow the identification of other phenomenon such as continuity and reaction restraint.

Performing the calculations for M in Section 6.1.2 brought to light the fact that the slab stiffness is quite substantial when compared to the stringers. In fact, the slabs would carry at least 50% and

40% of the section moments for Spans No. 1 and 2, respectively, if the sections were non-composite. Although the degree of composite action in these decks cannot be guaranteed, the above non-composite performance of the slab during the re-evaluation should at least be accounted for.

6.2/ Steel Floor Beams

Figures 22 and 23, and Figures 24 and 25 display the measured strains in the floor beams of Spans No. 1 and 2, respectively. These plots give the steel strains in the top and bottom flanges of the floor beams.

6.2.1/ Span No. 1

Figures 22 and 23 indicate that no apparent composite action exists between the concrete slab and the steel floor beams. The position of zero strain is very close to the neutral axis. These figures also depict the layout of the floor beams where the stringers sit on top of the floor beams separating them from the concrete slab.

There exists a notable difference in the strains for floor beams 3 and 4, as given in Figures 22 and 23, respectively. Floor beam 4 displays reasonably consistent pure bending responses with equal strains in the top and bottom. Floor beam 3 indicates higher compressive strains and lower tensile strains than floor beam 4. It is believed that this condition is due to the confinement at the ends of floor beam 3 imposed by its attachment to the trusses. Floor beam 4 is attached at the bottom of a member type 4, as given in Figure 2, where only that member and the bottom chord members frame in at the joint. Floor beam 3 is attached at the bottom of member type 6, as given in Figure 2, where five members frame into the joint, most of which are considerably stiffer than those at the previous joint. The stiffer condition at the ends of floor beam 3 would impose some resistance to rotation, but more resistance to the expansive movement of the bottom flange of the floor beam. In effect, this would simulate the application of an axial force applied below the neutral axis which would increase compressive strains and lower tensile strains as observed by the test results.

Using the strain results in Figure 22, the moments in the floor beams due to the measured strains were obtained. As an example, the moment in floor beam 4, under load level 6, with the vehicle in line 1, was about 182 kN-m. Using statics and assuming the stringers are simply supported between floor beams, the predicted moment is 244 kN-m. Obviously the concrete slab will carry some moment bending about its own axis. Using the effective slab width determined from the

analysis section of the OHBDC, the slab is estimated to carry at least 20% of the total moment. This would reduce the calculated moment to be carried by the naked beam to about 195 kN-m. This is closer to the experimental value of 184 kN-m, but still exceeds it by 6% which is minimal considering the assumptions made regarding the concrete slab performance (i.e., effective width and modulus). These calculations indicate that the contribution of the non-composite slab to the load carrying capacity of the floor beams should be considered in the final evaluation.

6.2.2/ Span No. 2

Figures 24 and 25 indicate that there is considerable composite action taking place between the concrete slab and the steel floor beams in Span No. 2. Unlike the layout for Span No. 1, this span has the stringers formed into the sides of the floor beams as shown in Figure 24. The concrete slab is in contact with the top flanges of the floor beams and some composition between the two is expected due to the high frictional forces between the surfaces induced under loading.

The method described in Section 6.1 provides the values for the experimental moments in the floor beams that are compatible with the analytical values. As an example, the calculated experimental moment for floor beam 2, under load level 5, with the vehicle in line 1, is approximately 282 kN-m. This assumes an effective slab width as determined by the analysis section of the OHBDC. The moment according to statics, assuming the deck is continuous over the floor beams as was determined in Section 6.1, is about 292 kN-m which is a difference of less than 4% from the experimental.

6.2.3/ Summary

Further discussion will be presented in Section 7 when the floor beams are re-evaluated. However, two points are clear with respect to the floor beams; the capacity of the floor beams should take into account the portion of moment carried by the slab; and the deck system should be assumed to be continuous over the floor beams when calculating the distribution of axle loads to the floor beams.

6.3/ Steel Trusses

Due to the slightly different configurations and testing procedures of the two steel trusses, they are presented separately.

6.3.1/ Span No. 1

The test results are presented as member axial forces derived from the average strains measured at a section. Only the results of line 3 and line 4, as identified in Figure 13, are given. Figures 26 through 39 and Figures 40 through 53 represent load levels 2 and 6, respectively. Each figure displays the axial force variation due to the test vehicle moving across the bridge in accordance with Figure 14. The figures also include the analytical results for the same loads using the original frame analysis discussed in Section 3.

The comparison of experimental and analytical results indicates that the frame analysis is a reasonably good model of the structural response of this truss, although some members require a more detailed discussion. The response comparisons were nearly the same for both load levels presented in the figures, therefore, the following discussion will refer only to load level 6. In addition, if a member does not appear in the subsequent discussion it means that the member response compared well with the analysis.

Member 1, the end diagonal, which is represented in Figure 40, appears to have less force than predicted, by about 18% at the maximum. It seems that the participation of the deck with the bottom chord, which will be discussed shortly, and restraint of bearings are responsible for this discrepancy. Member 4, in Figure 41, shows that the compressive force in the top chord compares well with the analysis.

The bottom chord responses, shown in Figures 42 and 43, demonstrate that a much lower member force exists than predicted by the analysis. This is a common observation in steel truss bridges where the deck system, through its connection to the trusses, shares the tensile forces with the bottom chord. A 50% reduction in the member forces is not uncommon.

The response of member 24, in Figure 45 was typical for the short vertical members to which the floor beams were attached. The general responses compared well, but there appeared to be a local "blip" in the analytical value taking it up to 20% over the experimental value. This occurred in position 4 where a heavily loaded axle was directly beside that member. It is important to note that this blip is very steep indicating it occurs if the load is placed just right and it goes directly into the panel point of the truss as a point load. Such point loads and accuracies of vehicle placement in testing do not occur in real life. However, it would be safer to use the analytical value in evaluation as the extent that this sharp peak may be rounded off by a local distribution of loads in the deck cannot be determined.

Member 30, shown in Figure 46, is typical of the internal diagonals. The general response compared favourably with the analysis, but usually fluctuate by ±20%. It is believed this fluctuation may be caused by the instrumentation which was not located on the neutral axis of the sections. Therefore, the axial strains cannot be isolated from any bending effects that the member may be subjected to.

Member 31, shown in Figure 47, appears to have 40% more force than predicted at its maximum. However, further investigation of the data showed that only the gauge on the inside was working. Invariably, the inside angle of double angle truss members pick up more tensile force near the bottom of the truss where the loads from the deck are transmitted into the truss. However, this is usually offset by the outside angle which takes less. This would then support the results displayed in Figure 47 where the discrepancy is more pronounced when the vehicle is close to the location where that member is attached to the bottom chord.

At first glance the response of member 33, shown in Figure 49, appears to be far from that predicted. It seems that one of the gauges was not working and in the initial transformations of data this was not noticed. The additional plot represents the single gauge that is believed to have been working properly and the latter provides a more respectable comparison with the analysis.

One last point should be mentioned regarding the reinforcement of some members, described in Section 2. The added steel sections, as shown in Figure 2, were invariably much larger than the original sections and were placed offset from the originals. Therefore, at the joints there were considerable eccentricities not accounted for in the frame analysis.

6.3.2/ Span No. 2

The test results for Span No. 2 are presented in a similar way with Figures 54 through 65 representing load level 1, and Figures 66 through 76, representing load level 5.

Unless mentioned otherwise the following discussion will be presented with respect to the results for load level 5.

Figure 66 shows that the response in the end diagonal was in good agreement with the model, whereas Figure 67 demonstrates that the top chord compressive force appears to be 20% below the predicted. There does not appear to be any clear reason why the latter discrepancy exists.

Figures 68 to 69 confirm again that the deck system through its attachment to the truss provides significant assistance to the bottom chord.

The internal diagonal members again demonstrate varying degrees of discrepancy with up to 30% difference from that predicted. Little confirmed reason can be provided for these differences other than the fact that the instrumentation was not placed at the neutral axis and therefore, the axial and bending components could not be separated. However, some additional instrumentation was provided for members 31 and 35 for load level 5, as shown in Figure 9.

The response of member 35, which was identified in the preliminary evaluation as being the critical member, is displayed in Figures 65 and 76 for load levels 1 and 5, respectively. According to this comparison, the analytical model closely matches the measured axial response for load level 1. However, in load level 5 the maximum calculated compressive force using the measured strains is about 30% below the analytical value. A closer look at the experimental data for member 35 is provided in Table 13. The gauge locations are as indicated in Figure 9 for Site 12 with gauges 1 and 2 on the top flanges. As the load is increased, the output of gauges 1 and 2 increases at first then begins to fall. This indicates some in-plane bending which creates tension in the top flanges thereby reducing the overall compression in those gauges. Only in load level 5 is this verified, as shown in Table 13, with the addition of the strip-chart recorder monitoring strains in the bottom. The calculated member forces using the measured strains, given in Figure 76, account for this bending component. Even though the member is bending it still resists a large axial compression, which may be the reason the analytical model still works reasonably well for the truss.

The two transducers, identified as Sites 21 and 22 in Figure 9, monitored in-plane displacement of members 35 and 31 for load level 5. These were employed to detect the in-plane movement of members during loading. These gauges indicated that very little displacement occurred at those points, with a maximum displacement for member 35 of less than 2 mm.

Due to the limited instrumentation of member 35 and the surrounding members it was not worthwhile to pursue this performance. However, particular attention will be given to this phenomenon when the next bridge with similar construction becomes available through MTO's bridge testing program.

6.3.3/ Summary

The testing of both trusses indicated that the frame analysis is a reasonable analytical model for light frame steel trusses. Although there are some apparent discrepancies in the previous member force comparisons, the majority of them can be explained as discussed in the previous sections. In future tests, it would appear to be very advantageous to instrument all sections such that the bending and axial forces in a member can be separated, however, this is not always possible due to the limited number of simultaneous monitoring channels available.

The transducers monitoring horizontal movement at the roller bearings of the two trusses provided maximum movements under the maximum loadings of only about 2 and 3 mm for Span No. 1 and 2, respectively . The analysis predicted movements of around 15 mm. Two factors would be influencing this movement: the interaction of the deck with the bottom chord in carrying tensile forces; and the possible reaction restraint at the support roller. In both bridges the transducers showed that the supports moved back to the original positions after the load was removed.

7/ Post-Testing Evaluation

7.1/ Steel Stringers

Although the test results indicated that some composite action exists between the concrete deck and the stringers in both spans this cannot be assured beyond the maximum applied load. The first step in the re-evaluation was to use the maximum applied proof load as a means of determining a scale down factor. This is performed by determining the maximum moment in the stringers due to the maximum applied proof load, M_t, and the factored OHBDC design load, M_d. The maximum proof load moment applied in the test, M_t, can now be considered equivalent to the moment capacity of the stringer that is available for live load. The dead load effect is already being carried by the stringer. Therefore, the equation for the scale down factor F becomes:

$$F = \frac{\phi M_n - \text{Moment due to factored dead load}}{\text{Moment due to factored live load}} = \frac{M_t}{M_d}$$

Applying this approach to both decks provides scale down factors based on the proof loading of 0.77 for both spans. This represents an improvement over the preliminary evaluation for Span No. 1 but not for Span No. 2, where the F factors were 0.72 and 0.78, respectively, as given in Section 3. However, a more detailed application of the test results can be performed based on the discussions from Section 6.

7.1.1/ Span No. 1

A summary of the discussions presented in Section 6.1 for the steel stringers in Span No. 1 is presented below:

1. The concrete slab will carry at least 50% of the section moment under non-composite action.

2. The deck system demonstrates some continuity over the floor beams thereby reducing the maximum moments.

3. The apparent composite action between the concrete slab and the stringers cannot be assured beyond the maximum applied proof load.

4. Using the tensile steel strains from the stringers indicates the distribution performance is probably worse than that given by the OHBDC for a non-composite deck.

The single most effective point is that the slab will carry 50% of the section moment which reduces the applied moment to the steel section by half. Even if the effects of deck continuity and composite action are discounted and the worst distribution width from Table 7 is applied, the new capacity reduction factor will still be over 1.0. In fact, even with a 10% reduction in strength for deterioration the capacity reduction factor is 1.22. Therefore, it is recommended that the stringer system for this span not be posted for any load restrictions.

7.1.2/ Span No. 2

The discussions from Section 6.1 for the stringers of Span No. 2, are summarized below:

1. The concrete slab carries at least 40% of the section moment under non-composite action.

2. The deck system is fully continuous over the floor beams thereby reducing the maximum moment.

3. The test results indicate that the distribution width is close to that given by the OHBDC for a non-composite deck system.

4. The apparent composite action cannot be assured beyond the maximum applied proof load.

In re-evaluating the stringers for this span we assumed the slab will carry 40% of the section moment and also accounted for the continuity of the deck system over the floor beams. Taking a 10% reduction in strength for deterioration and using the worst distribution width from Table 11, the new capacity reduction factor is 1.69. Therefore the stringers of this span do not require any posting.

7.2/ Floor Beams

The capacity reduction factors for the floor beams were obtained in a manner similar to that applied to the stringers. Based on the proof loading, the capacity reduction factors were 0.79 and 0.77 for the floor beams of Spans No. 1 and 2, respectively. When compared to the preliminary evaluation values of 0.57 and 0.87 for Spans No. 1, and No. 2, respectively, an improvement is noted only for Span No. 1.

A more detailed evaluation using the test results can now be performed.

7.2.1/ Span No. 1

Looking at the test results as presented in Section 6.2 there are a number of points which need to be considered:

1. The deck slab will carry up to 20% of the section moment assuming an effective slab width as specified in the OHBDC.

2. Continuity of the deck system over the floor beams will increase the distribution of wheel loads to the floor beams by about 7% based on the critical OHBDC axle configuration.

Taking into consideration these points and accounting for a 10% strength reduction for deterioration, a new capacity reduction factor of 0.76 is obtained. Therefore, the controlling factor will be 0.79 as determined from the proof loading, and the posting for the floor beams in Span No. 1 should be based on this value.

7.2.2/ Span No. 2

Following the same approach for the floor beams of Span No. 2, the following points should be considered:

1. The deck slab will carry up to 15% of the section moment assuming an effective slab width as specified in the OHBDC.

2. Continuity of the deck system over the floor beams will increase the distribution of wheel loads to the floor beams by about 8% based on the critical OHBDC axle configuration.

3. The apparent composite action of the concrete slab and floor beams cannot be assured beyond the applied proof loading.

Taking into consideration the first two points and accounting for a 10% strength reduction for deterioration, the new capacity reduction factor is 0.85. This represents a slight reduction over that obtained in the preliminary evaluation, but then the continuity of the deck system and a reduction for deterioration were not considered for the floor beams in Section 3. It is recommended that F= 0.85 be used to determine the posting requirements for the floor beams in Span No. 2.

7.3/ Steel Trusses

An extensive post-testing evaluation was not considered necessary for the steel trusses. The evaluation of the test results in Section 6.3 indicated that the frame analysis used in the preliminary evaluation was a reasonably good analytical model.

7.3.1/ Span No. 1

A review of the preliminary evaluation for this truss, as given in Table 1, indicates that member 30 was the critical member. Figure 46 compares axial force determined from the test results to predicted force using the analytical model. However, the model overestimates the member force making it more conservative. It is not considered favourable to take advantage of this apparent overestimation since the accuracy of the axial force determined from the test results is questionable. This is due to the instrumentation of these members which did not allow for the separation of bending and axial components in the members.

The governing capacity reduction factors for this truss are to be taken from Table 1 for member 30.

7.3.2/ Span No. 2

Reviewing the preliminary evaluation for this truss, as given in Table 2, identifies member 29 as the critical member after the compressive capacity of member 35 was ignored. Again the test results indicate that the model apparently overestimates the force in this member. However, for the

same reasoning as for Span No. 1, it was decided that the governing capacity reduction factor be taken from Table 2 for member 29 without any adjustment.

7.4/ Summary

Reviewing the post-testing evaluation, a new set of governing capacity reduction factors for the various bridge components is now available. These are given in Table 14. Final recommended postings are given in the following section.

8/ Conclusions and Recommendations

Table 14 summarizes the governing capacity reduction factors from the post-testing evaluation. Using the effective spans for each member a posting was determined for each F value according to the OHBDC. This resulted in a new triple posting of 18, 32 and 36 tonnes for evaluation levels 1, 2 and 3, respectively. This is a marked improvement over the posting that would have resulted from the preliminary evaluation of approximately 10, 15 and 25 tonnes.

This new posting assumes that any seriously deteriorated members will be repaired or replaced. However, it does not require any additional strengthening. Since the live load factor used in the investigation was 1.25, the bridge will need to be re-evaluated in 5 years.

It is recommended that in the future care be taken when instrumenting truss members in order to better isolate the axial forces from bending effects. In addition, both the top and bottom flanges of steel beams across a deck section should be instrumented. The latter would allow better evaluation of the test results to determine the load distribution characteristics.

References

[1] "Ontario Highway Bridge Design Code," Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1983.

[2] "ICES STRUDL II - Structural Design Language," Massachusetts Institute of Technology, Cambridge, Massachusetts, 1972.

[3] Bakht, B. "Testing of an Old Short Span Slab-On-Girder Bridge", Ontario Ministry of Transportation, SRR-88-01, Downsview, Ontario, 1988.

TABLES

and

FIGURES

Member #	Factored Member Resistance (kN)	Factored Dead Load (kN)	Maximum Factored Live Load OHBDC (kN)			Scale Down Factor F OHBDC		
			Level I	Level II	Level III	Level I	Level II	Level III
1	-1823	-493	-524_1	-657_1	-732_1	2.54	2.02	1.82
4	-2023	-575	-623_1	-780_1	-868	2.32	1.86	1.67
9	949	347	371_1	465_1	519_1	1.62	1.29	1.16
13	1580	543	575_1	712_1	814_1	1.80	1.46	1.27
21	824	183	249_1	279	337	2.57	2.30	1.90
24	416	60	243	243	243	1.47	1.47	1.47
30	474	184	258	294	351	1.12	0.99	0.83
31	474	142	248	252	312	1.34	1.32	1.06
32	-90.4	77_2	-55	-67	-67	3.04	2.50	2.50
	392	90	199	231	252	1.52	1.31	1.20
39	-1775	-126	-201_1	-235	-311	8.20	7.02	5.30
40	-1775	-85	-192	-230	-250	8.80	7.35	6.76
	2108	-72_2	61	70	71	35.7	31.1	30.7
41	-1155	-45	-138_1	-171	-223	8.04	6.49	4.98
	1326	-38_2	131	142	149	10.4	9.61	9.15

1 - Lane Load Governs (Class C) 2 - All Dead Load Factors = 1.0

Table 1/ Preliminary OHBDC Evaluation - Span No. 1, Truss

Member #	Factored Member Resistance (kN)	Factored Dead Load (kN)	Maximum Factored Live Load OHBDC (kN)			Scale Down Factor F OHBDC		
			Level I	Level II	Level III	Level I	Level II	Level III
1	-2051	-413	-468_1	-586_1	-664	3.50	2.80	2.47
5	-1680	-480	-536_1	-681_1	-768	2.24	1.76	1.56
10	935	267	302_1	370_1	429	2.21	1.81	1.56
14	1418	471	501_1	636_1	718	1.89	1.49	1.32
19	349	71.0	265	265	265	1.05	1.05	1.05
27	614	187	252_1	318	362	1.69	1.34	1.18
28	-72.7	89.4_2	-123	-133	-133	1.32	1.22	1.22
	462	107	193_1	254	282	1.84	1.40	1.26
29	-91.2	28.0_2	-74.7	-89.0	-90.3	1.60 $(0.90)_3$	1.34 $(0.75)_3$	1.32 $(0.74)_3$
	392	33.8	116_1	155	170	3.09	2.31	2.11
31	-85.2	10.2_2	-64.9	-91.6	-95.2	1.47	1.05	1.00
	392	12.5	106	137	149	3.58	2.77	2.55
35	-46.2	-15.1	-71.6_1	-99.6	-110	0.43	0.31	0.28
	267	-12.9_2	79.6	95.2	98.7	3.52	2.94	2.84

1 - Lane Load Governs (Class C) 2 - All Dead Load Factors = 1.0 3 - Re-analysis with member 35 removed

Table 2/ Preliminary OHBDC Evaluation - Span No. 2, Truss

Minimum
Scale Down Factor F
OHBDC Evaluation

	Level I	Level II	Level III
Span No. 1			
Truss	1.12	0.99	0.83
Stringers	0.72	0.72	0.72
Floor Beams	0.57	0.57	0.57
Span No.2			
Truss	0.90	0.75	0.74
Stringers	0.78	0.78	0.78
Floor Beams	0.87	0.87	0.87

Table 3/ Summary of Scale Down Factors - Preliminary

	Load Level	Total Load on Rear Tandem (kN) (Axle No. 4 & 5)	Total Load on Front Tandem (kN) (Axle No. 2 & 3)	Steering (kN) Axle No. 1	Total (kN)
Vehicle No. 1	1	180	177	48	405
	2	223	225	50	498
	3	251	252	52	555
	4	268	271	53	592
	5	316	315	55	686
	6	362	359	57	778
Vehicle No. 2	1	179	167	49	395
	2	223	212	53	488
	3	251	238	56	545
	4	270	255	57	582
	5	318	296	61	675
	6	365	338	65	768

Table 4/ Test Loading for Span No. 1

	Load Level	Total Load on Rear Tandem (kN) (Axle No. 4 & 5)	Total load on Front Tandem (kN) (Axle No. 2 & 3)	Steering (kN) Axle No. 1	Total (kN)
Vehicle No. 1	1	180	177	48	405
	2	223	225	50	498
	3	268	271	53	592
	4	316	315	55	686
	5	362	359	57	778
Vehicle No. 2	1	179	167	49	395
	2	223	212	53	488
	3	270	255	57	582
	4	300	279	59	638

Table 5/ Test Loading for Span No. 2

Load Level	Line Position	Steel Strains from North to South Side 1	2	3	4	5	6
2	1	43	119	154	176	126	52
2	2	66	133	160	176	105	28
2	3	82	136	154	158	96	25
4	1	49	150	208	198	164	61
4	2	79	163	202	184	114	30
4	3	108	179	209	181	121	30
6	1	74	226	315	281	231	94
6	3	151	254	298	244	173	39

Table 6/ Experimental Strains in Steel Stringers (Span No. 1, Long. Position 3)

Load Level	Line Position	Sum of Strains	Maximum Load Fraction	Experimental Distribution Width (m)
2	1	670	0.263	1.74
2	2	668	0.263	1.73
2	3	651	0.243	1.88
4	1	830	0.251	1.82
4	2	772	0.262	1.74
4	3	828	0.252	1.81
6	1	1221	0.258	1.77
6	3	1159	0.257	1.78

Table 7/ Experimental Distribution Widths (Span No. 1, Long. Position 3)

Load Level	Line Position	Steel Strains from North to South Side 1	2	3	4	5	6
1	1	47	78	94	86	73	46
1	2	73	91	90	76	54	14
1	3	118	123	120	83	53	24
3	1	113	173	194	170	130	106
3	2	145	195	196	159	101	107
3	3	179	205	196	142	87	52
5	1	159	239	273	250	194	150
5	2	228	280	281	238	148	102
5	3	283	304	292	231	134	79

Table 8/ Experimental Strains in Steel Stringers (Span No. 2, Long. Position 1)

Load Level	Line Position	Sum of Strains	Maximum Load Fraction	Experimental Distribution Width (m)
1	1	424	0.222	2.06
1	2	398	0.229	2.00
1	3	521	0.236	1.93
3	1	886	0.219	2.08
3	2	903	0.217	2.10
3	3	861	0.238	1.92
5	1	1265	0.216	2.12
5	2	1277	0.220	2.07
5	3	1323	0.230	1.99

Table 9/ Experimental Diistribution Widths (Span No. 2, Long. Position 1)

Load	Line	Flange	From North to South Side (microstrain)					
3	1	Top	-5	24	90	30	23	-42
		Bottom	113	173	194	170	130	106
3	2	Top	-3	13	64	16	2	-24
		Bottom	145	195	196	159	101	107
3	3	Top	-6	7	51	7	3	-27
		Bottom	179	205	196	142	87	52
5	1	Top	14	40	110	56	43	-63
		Bottom	159	239	273	250	194	150
5	2	Top	49	68	131	68	33	-24
		Bottom	228	280	281	238	148	102
5	3	Top	84	108	143	82	29	-19
		Bottom	283	304	292	231	134	79

Table 10/ Experimental Steel Strains Steel Stringers (Span No. 2, Long. Position 1)

Load	Line	Moment in Each Stringer kN-m From North to Side Side of Deck 1	2	3	4	5	6	Total Section Moment (kN-M)	Distribution Width (m)
3	1	18	27	29	26	20	18	138	2.17
3	2	23	31	30	25	16	18	142	2.09
3	3	29	32	30	22	14	9	136	1.94
5	1	25	37	41	38	30	25	197	2.19
5	2	35	43	42	36	23	17	195	2.07
5	3	43	46	43	35	21	13	200	1.98

Table 11/ Experimental Section Moments for Stringers in Span No. 2

Load	Line	Total Section Moment Across Width of Deck (kN-m)		
		Experimental	Analytical	
			Simply Supp.	Continuous
3	1	138	261	137
3	2	142	261	137
3	3	136	261	137
5	1	197	345	185
5	2	195	345	185
5	3	200	345	185

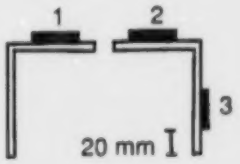
Table 12/ Total Section Moment: Analytical vs. Experimental

Measured Strains* (με)

Load Level	Site No. 1	Site No. 2	Site No. 3 (strip chart recorder)
1	-127	-120	—
2	-137	-130	—
3	-151	-153	—
4	-131	-162	—
5	-96	-147	-250

* Results for line 3, transverse position 10

Gauge sites



(see also site 12, 14, Figure 9)

Table 13/ Test Results — Steel Truss, Span No. 2, Member 35

Minimum
Scale Down Factor F
Post-testing Evaluation

	Level I	Level II	Level III	Effective Span (m) of Component
Span No. 1				
Truss	1.12	0.99	0.83	48.8
Stringers	1.22	1.22	1.22	4.06
Floor Beams	0.79	0.79	0.79	5.33
Span No. 2				
Truss	0.90	0.75	0.74	45.7
Stringers	1.69	1.69	1.69	5.08
Floor Beams	0.85	0.85	0.85	5.33

Table 14/ Summary of Scale Down Factors - Post-testing Evaluation

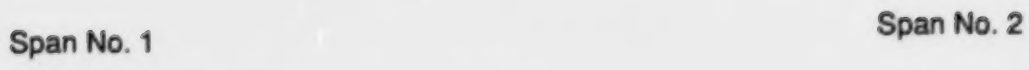


Figure 1/ Turner's Bridge, Site #2-066

Span No. 1

Span No. 2

Figure 1/ Turner's Bridge, Site #2-066

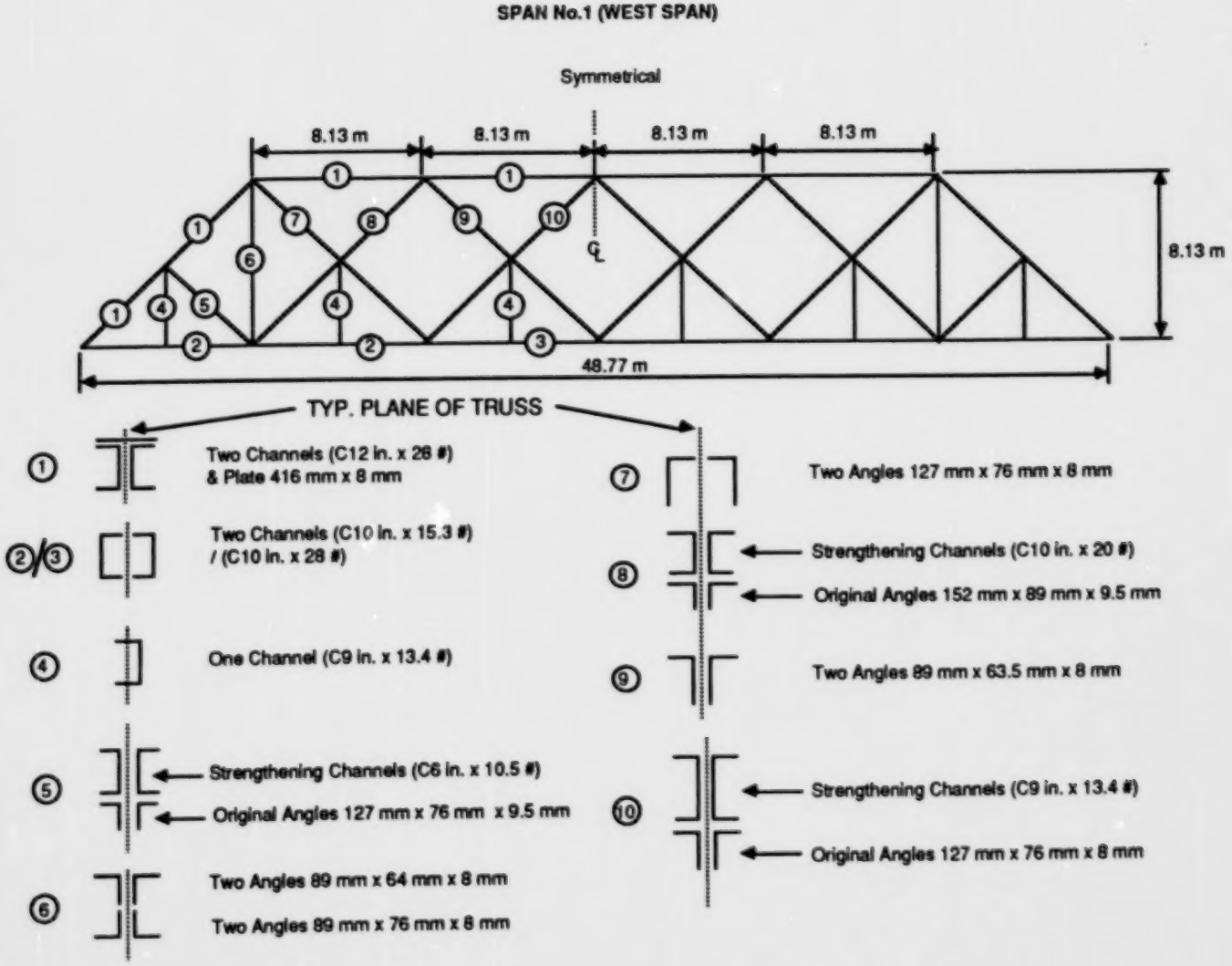

SPAN No.1 (WEST SPAN)
Symmetrical
8.13 m
8.13 m
8.13 m
8.13 m
8.13 m
48.77 m
TYP. PLANE OF TRUSS
1 Two Channels (C12 in. x 28 #) & Plate 416 mm x 8 mm
2/3 Two Channels (C10 in. x 15.3 #) / (C10 in. x 28 #)
4 One Channel (C9 in. x 13.4 #)
5 Strengthening Channels (C6 in. x 10.5 #)
Original Angles 127 mm x 76 mm x 9.5 mm
6 Two Angles 89 mm x 64 mm x 8 mm
Two Angles 89 mm x 76 mm x 8 mm
7 Two Angles 127 mm x 76 mm x 8 mm
8 Strengthening Channels (C10 in. x 20 #)
Original Angles 152 mm x 89 mm x 9.5 mm
9 Two Angles 89 mm x 63.5 mm x 8 mm
10 Strengthening Channels (C9 in. x 13.4 #)
Original Angles 127 mm x 76 mm x 8 mm

Figure 2/ Section Properties & Dimensions–Truss Span No.1

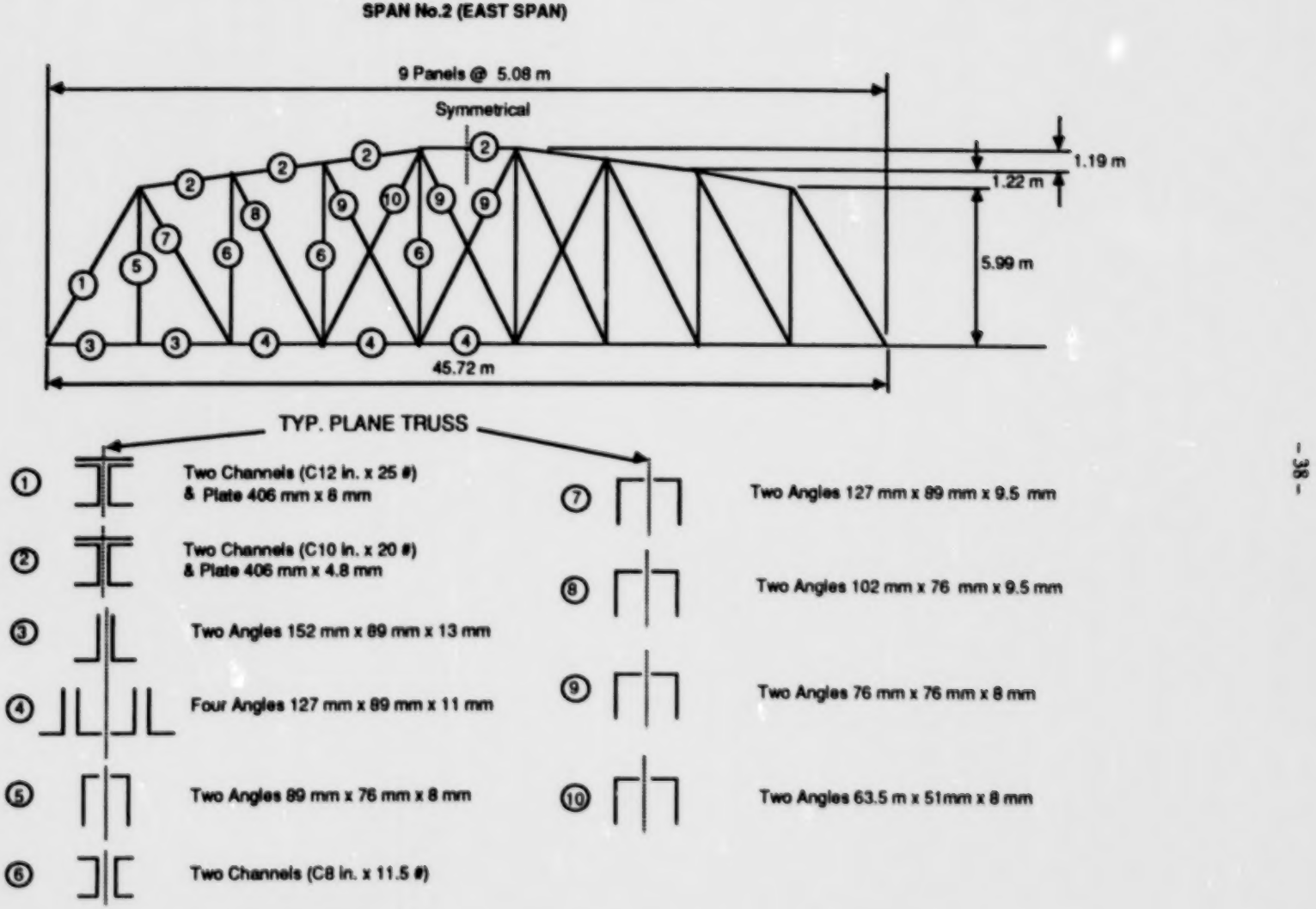
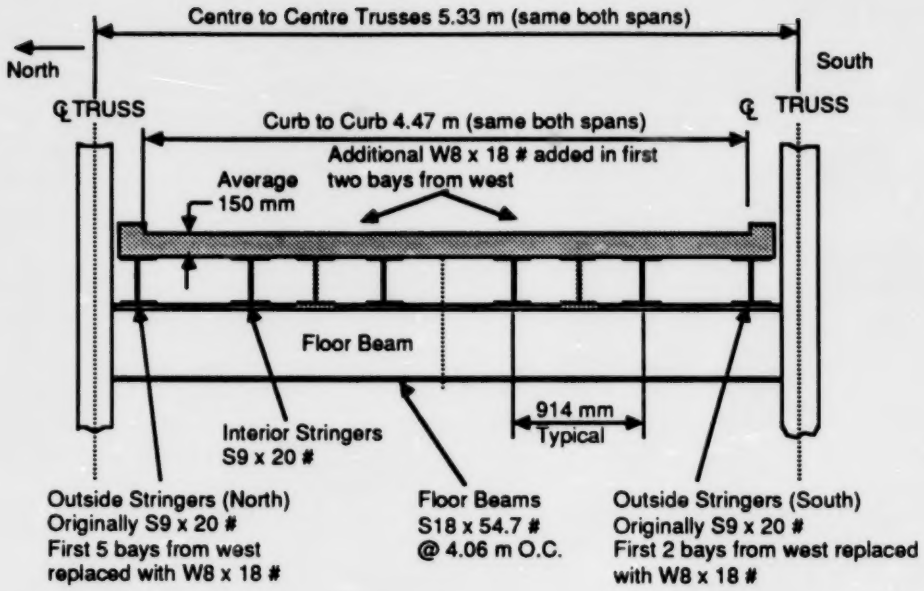
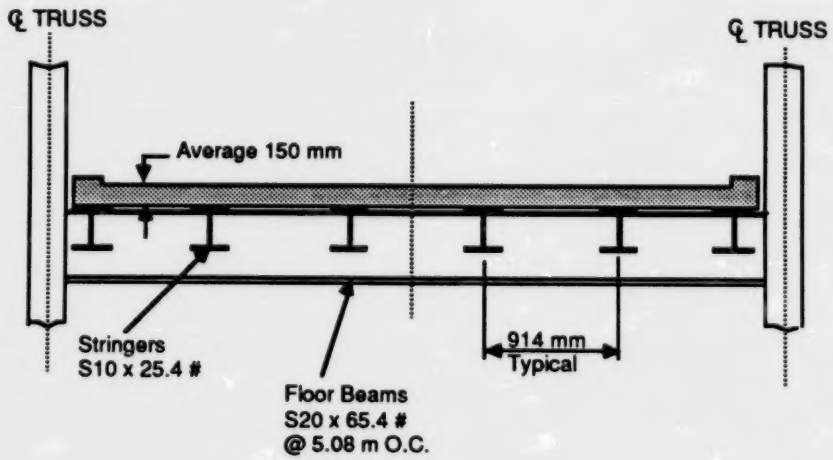

SPAN No.2 (EAST SPAN)
9 Panels @ 5.08 m
Symmetrical
1.19 m
1.22 m
5.99 m
45.72 m
TYP. PLANE TRUSS
① Two Channels (C12 in. x 25 #) & Plate 406 mm x 8 mm
② Two Channels (C10 in. x 20 #) & Plate 406 mm x 4.8 mm
③ Two Angles 152 mm x 89 mm x 13 mm
④ Four Angles 127 mm x 89 mm x 11 mm
⑤ Two Angles 89 mm x 76 mm x 8 mm
⑥ Two Channels (C8 in. x 11.5 #)
⑦ Two Angles 127 mm x 89 mm x 9.5 mm
⑧ Two Angles 102 mm x 76 mm x 9.5 mm
⑨ Two Angles 76 mm x 76 mm x 8 mm
⑩ Two Angles 63.5 m x 51mm x 8 mm

Figure 3/ Section Properties & Dimensions–Truss, Span No. 2

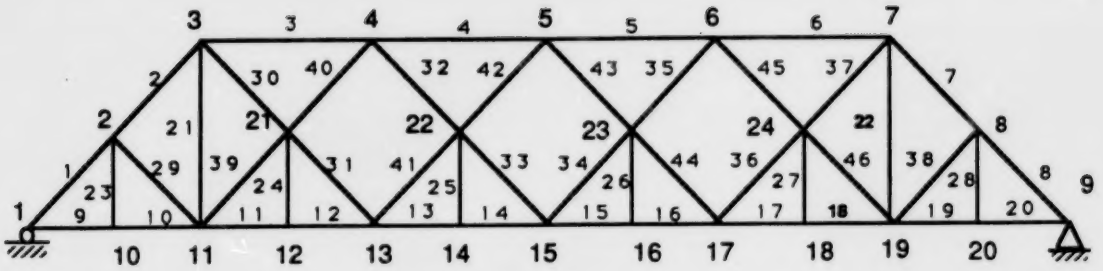

Centre to Centre Trusses 5.33 m (same both spans)
North
South
℄ TRUSS
℄ TRUSS
Curb to Curb 4.47 m (same both spans)
Additional W8 x 18 # added in first two bays from west
Average 150 mm
Floor Beam
914 mm Typical
Interior Stringers S9 x 20 #
Outside Stringers (North) Originally S9 x 20 # First 5 bays from west replaced with W8 x 18 #
Floor Beams S18 x 54.7 # @ 4.06 m O.C.
Outside Stringers (South) Originally S9 x 20 # First 2 bays from west replaced with W8 x 18 #

Cross-Section Span No. 1

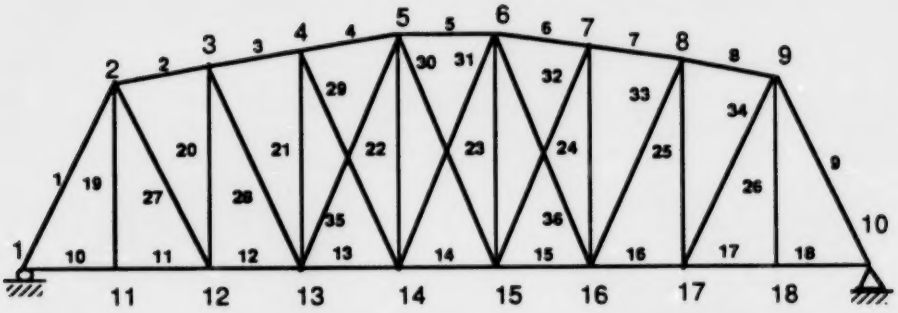


Cross-Section Span No. 2

Figure 4/ Section Properties and Dimensions – Deck Systems



TRUSS - SPAN No. 1



TRUSS - SPAN No. 2

Figure 5/ Schematic Representation of Truss for Frame Analysis

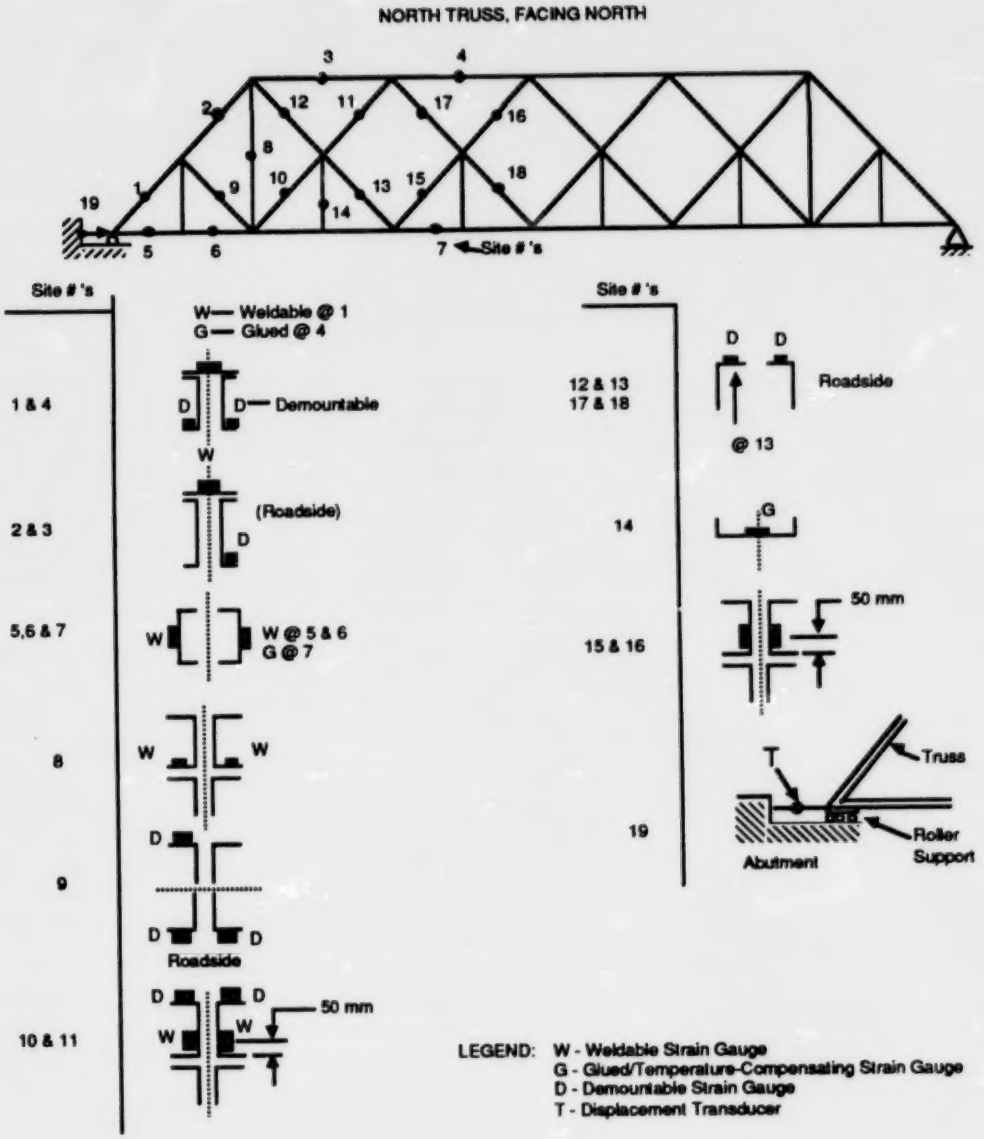
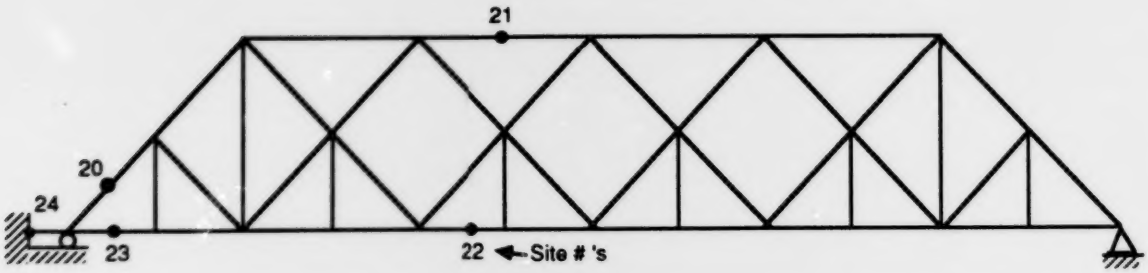


Figure 6/ Instrumentation Span No. 1 North Truss

SOUTH TRUSS, FACING NORTH



Site # 's	
20 & 21	Same as 2 & 3 (See Figure 6)
22 & 23	Roadside
24	Same as 19 (See Figure 6)

LEGEND: W - Weldable Strain Gauge
G - Glued/Temperature-Compensating Strain Gauge
D - Demountable Strain Gauge
T - Displacement Transducer

Figure 7/ Instrumentation Span No. 1 South Truss

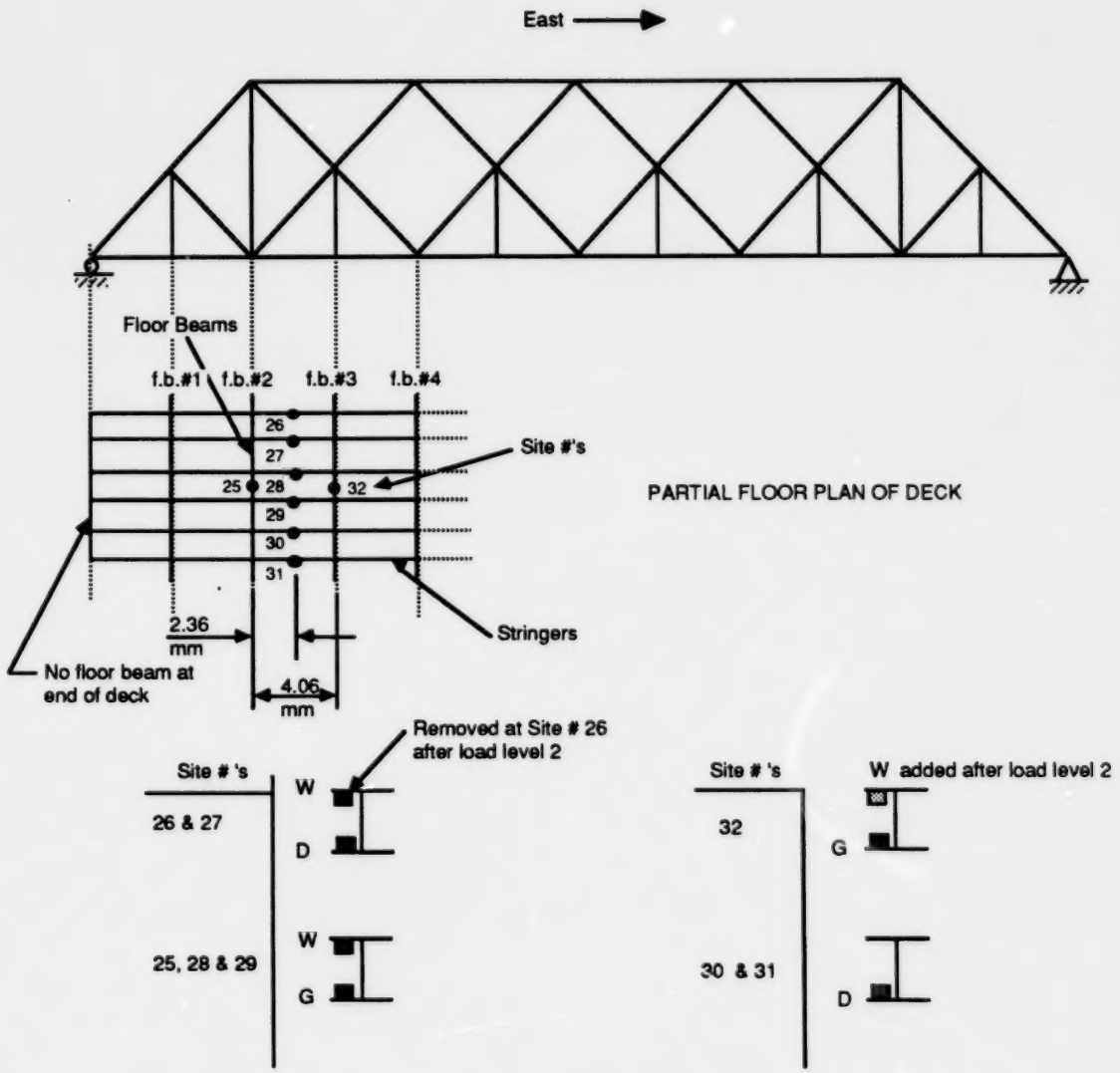
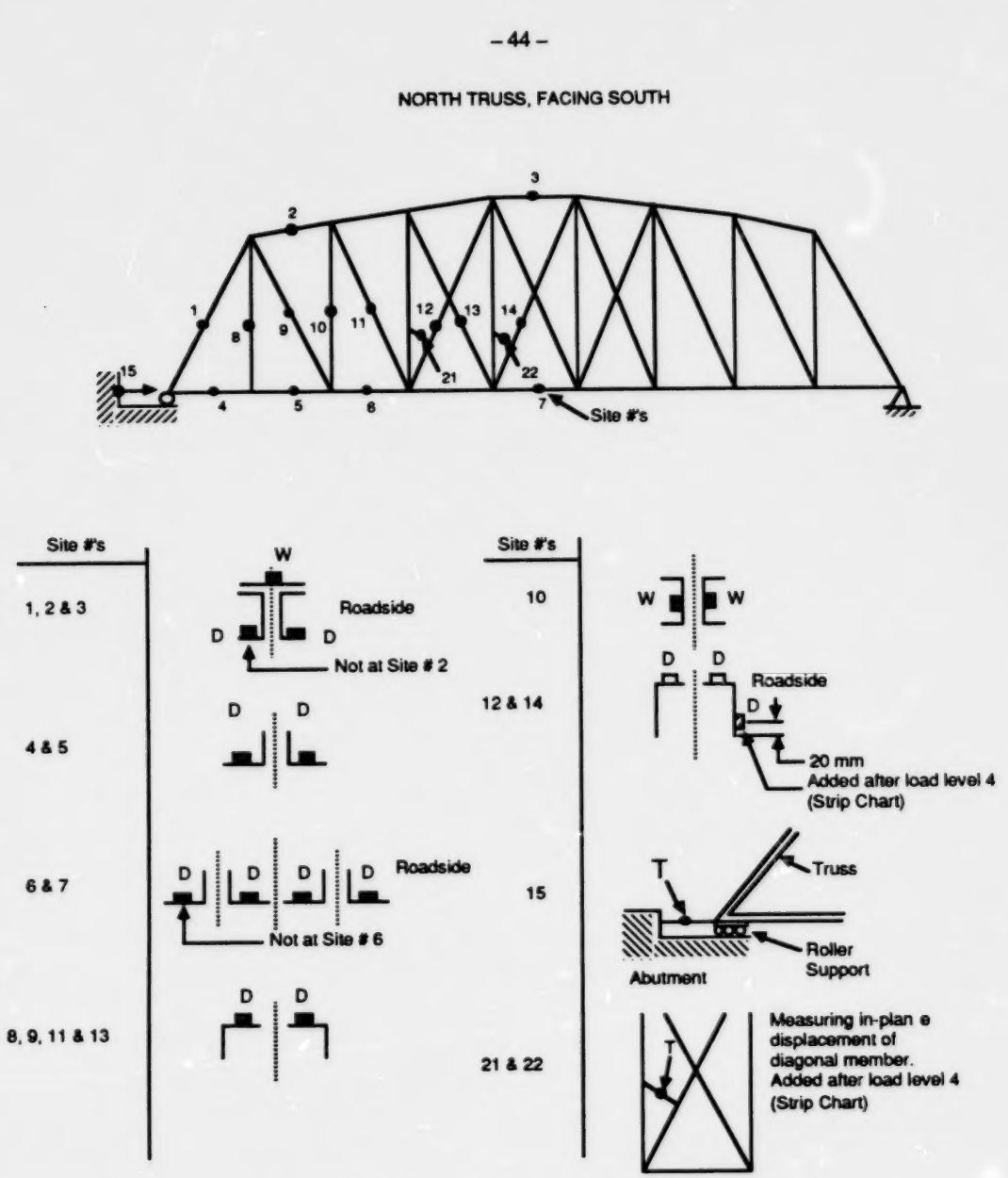


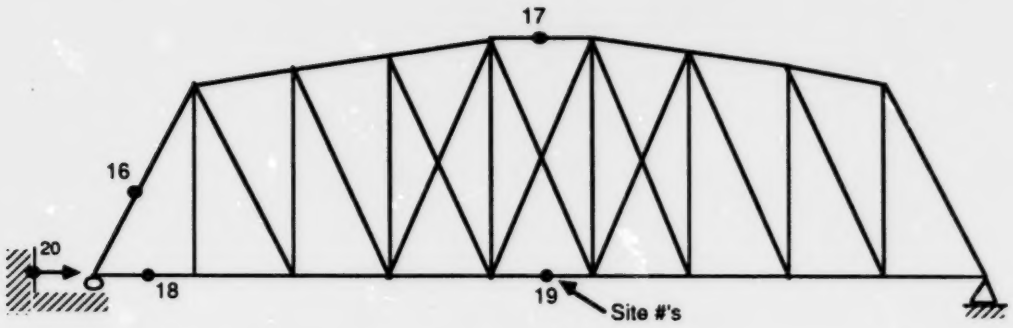
Figure 8/ Instrumentation Span No. 1 Deck System

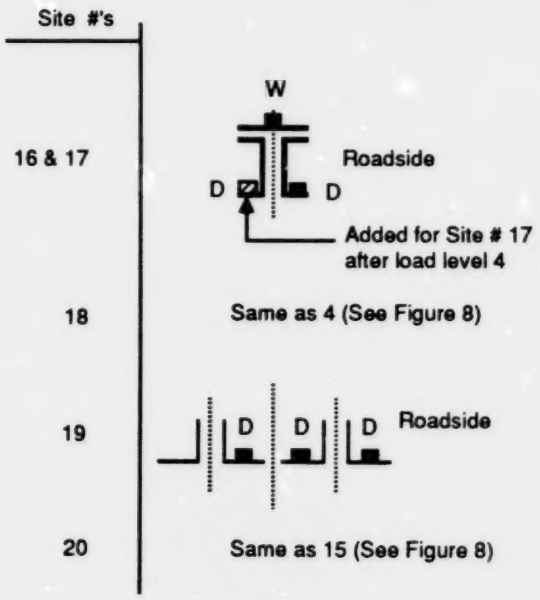


LEGEND: W - Weldable Strain Gauge
G - Glued/Temperature-Compensating Strain Gauge
D - Demountable Strain Gauge
T - Displacement Transducer

Figure 9/ Instrumentation Span No. 2 North Truss

SOUTH TRUSS, FACING SOUTH





LEGEND: W - Weldable Strain Gauge
G - Glued/Temperature-Compensating Strain Gauge
D - Demountable Strain Gauge
T - Displacement Transducer

Figure 10/ Instrumentation Span No. 2 South Truss

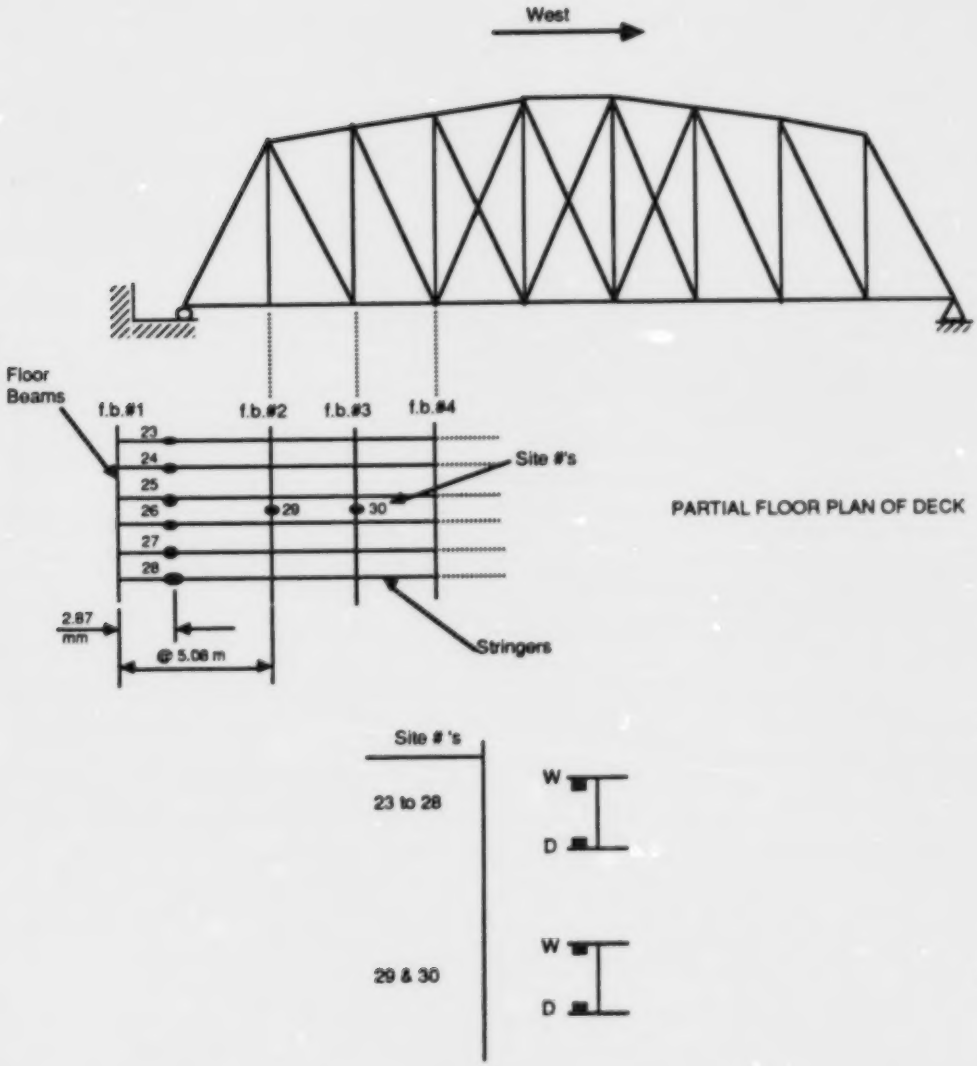
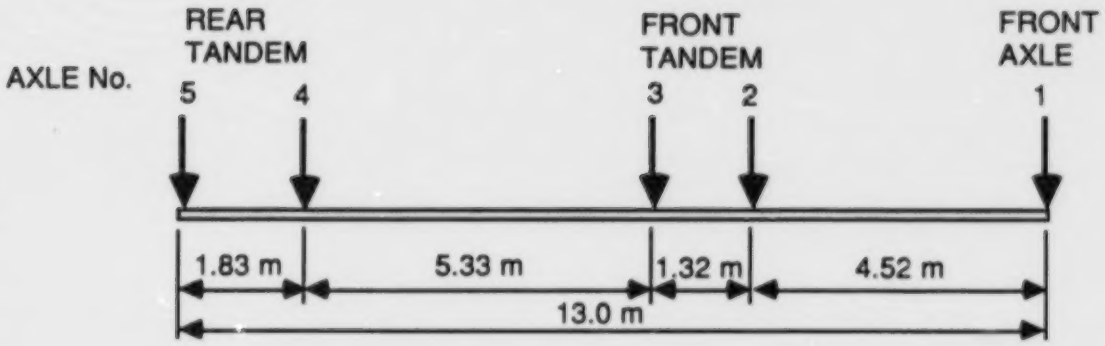
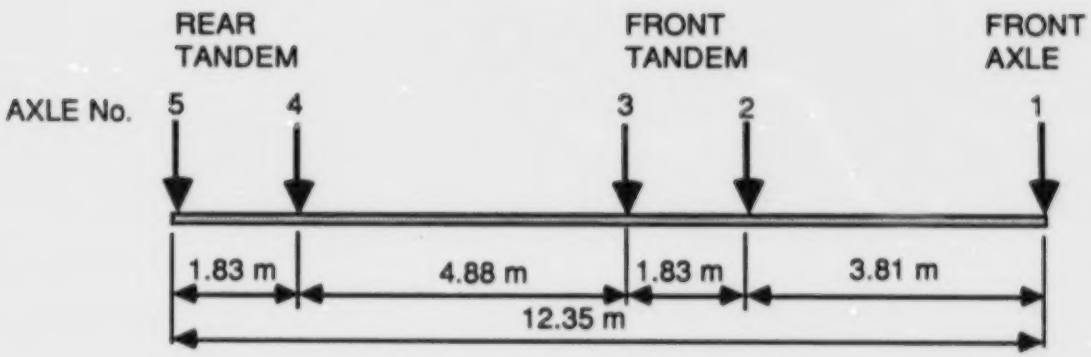


Figure 11/ Instrumentation Span No. 2 Deck System

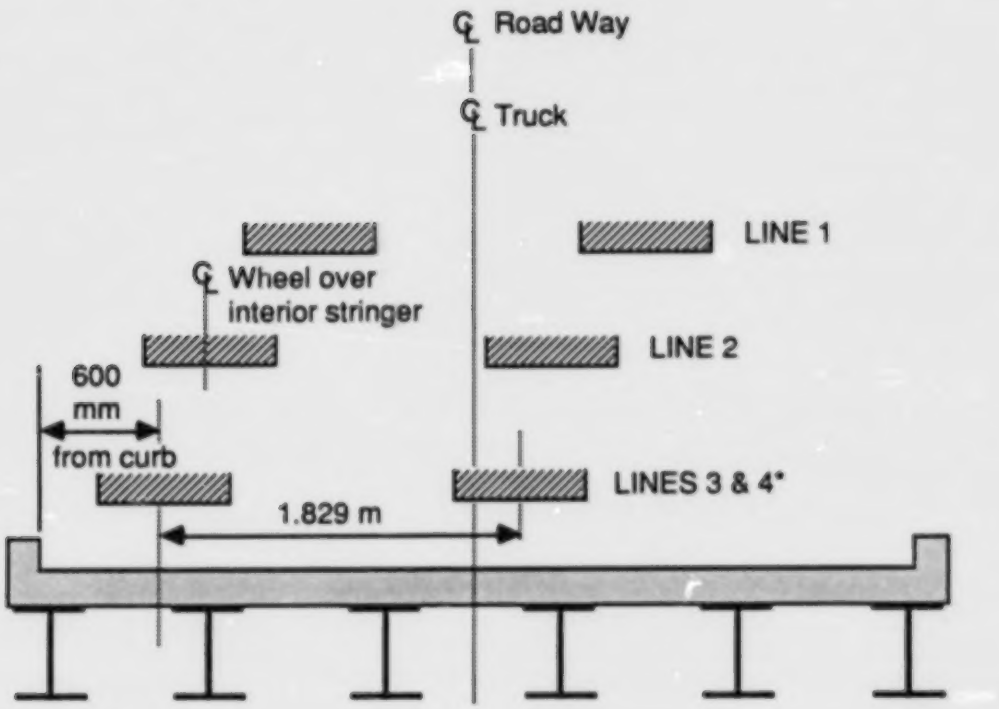


AXLE CONFIGURATION OF TEST VEHICLE No. 1, "KENWORTH"



AXLE CONFIGURATION OF TEST VEHICLE No. 2, "MAC"

Figure 12/ Axle Configuration of Test Vehicles



* Line 4 is the same as Line 3 with Vehicle No. 2 following at a clear distance of 10 m

Figure 13/ Test Vehicle Placement–Line Position

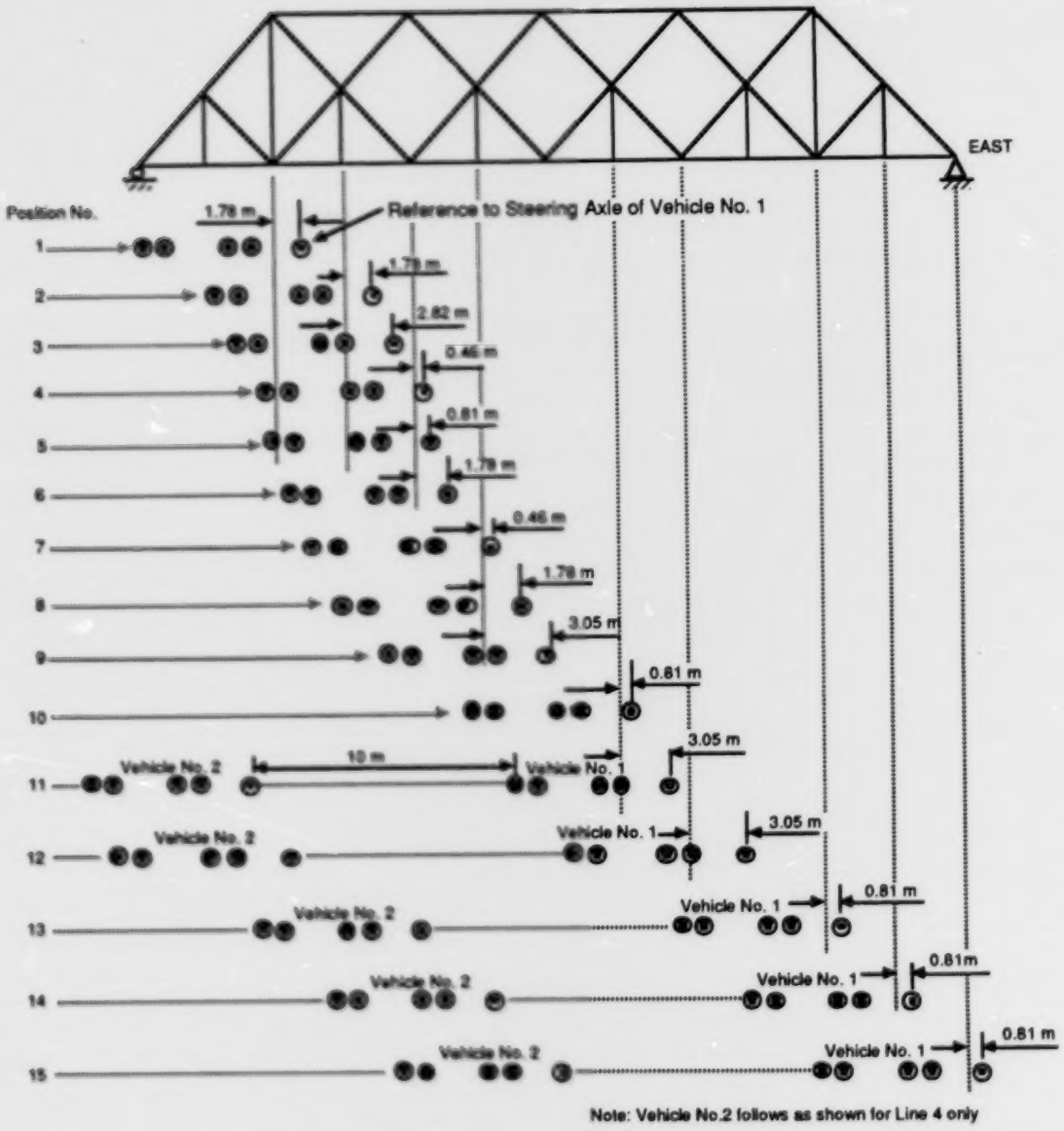
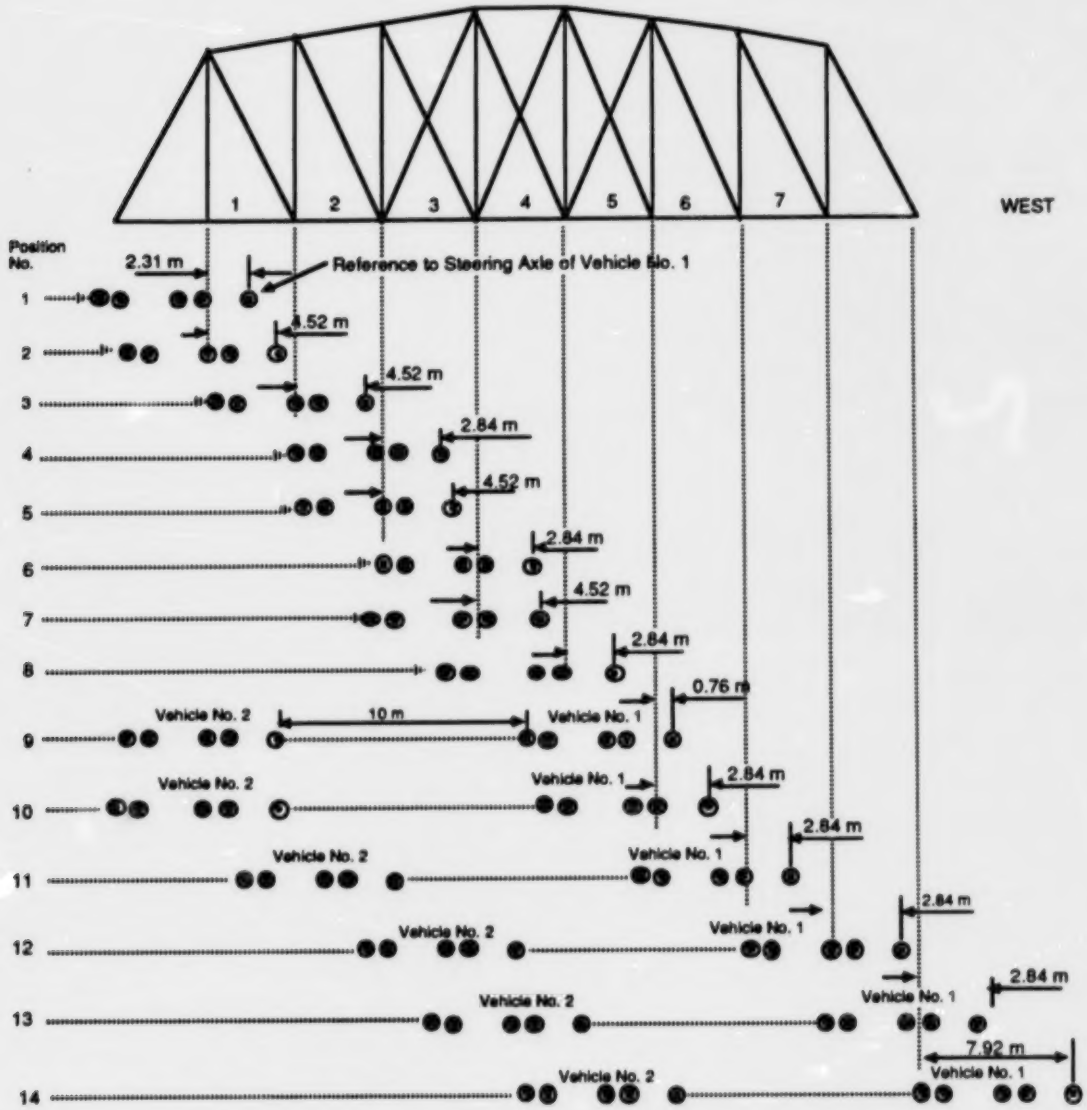


Figure 14/ Test Vehicle Placement–Longitudinal Positions, Span No. 1



Note: Vehicle No. 2 follows as shows for Line 4 only

Figure 15/ Test Vehicle Placement–Longitudinal Positions, Span No. 2

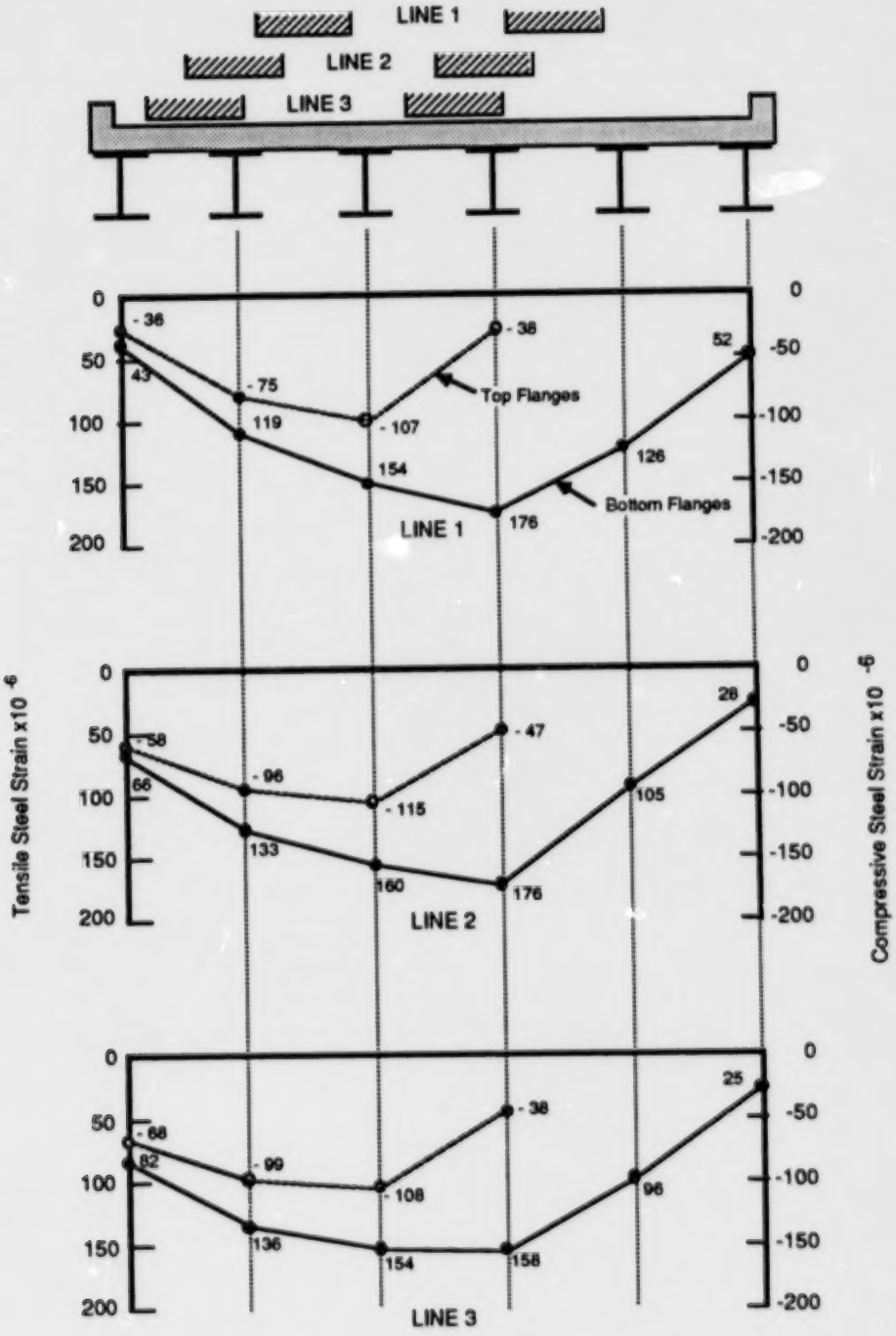


Figure 16/ Test Results–Steel Stringer Strains, Span No. 1, Load Level 2, Longitudinal Position 3

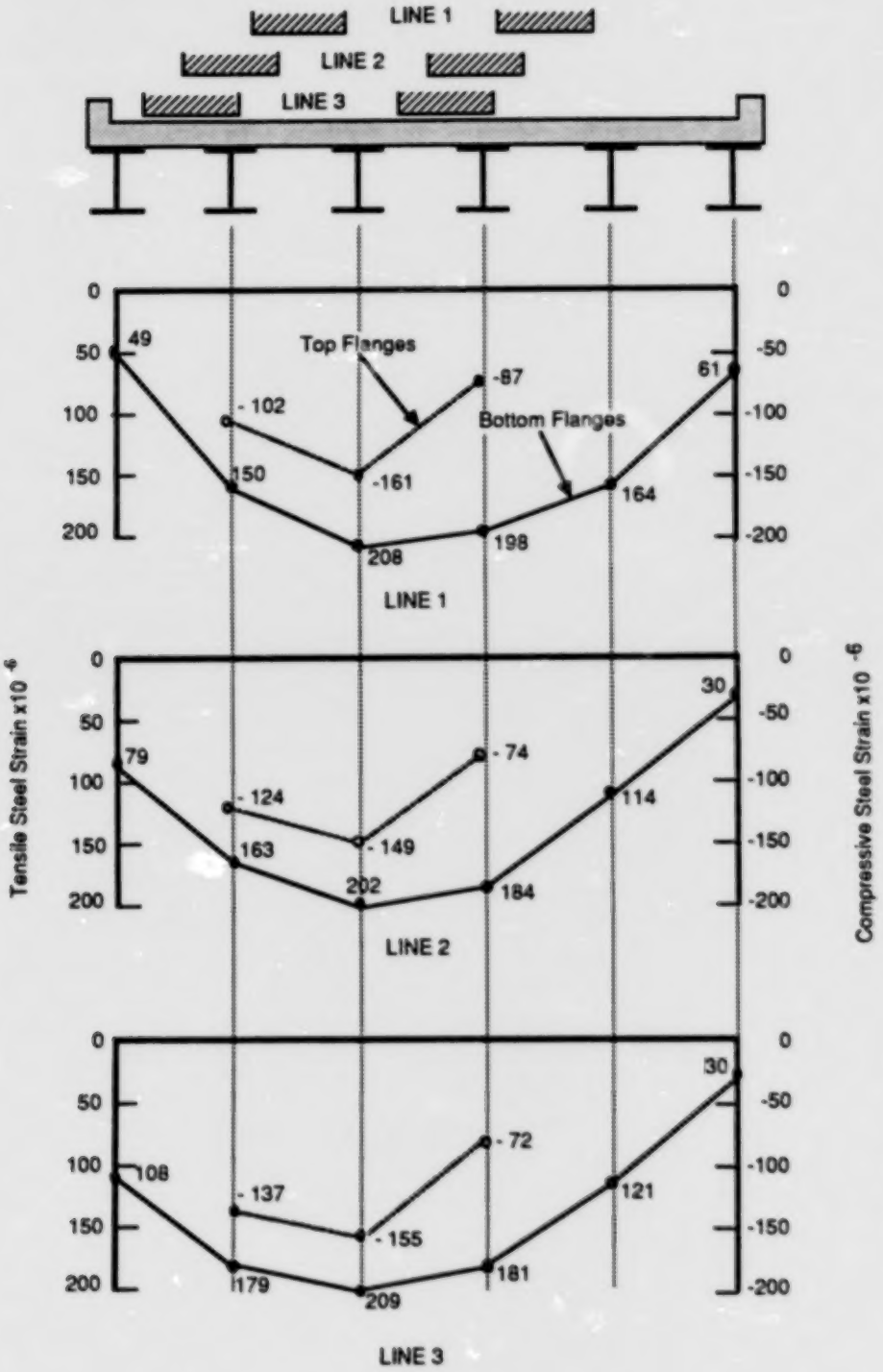


Figure 17/ Test Results–Steel Stringer Strains, Span No. 1, Load Level 4, Longitudinal Position 3

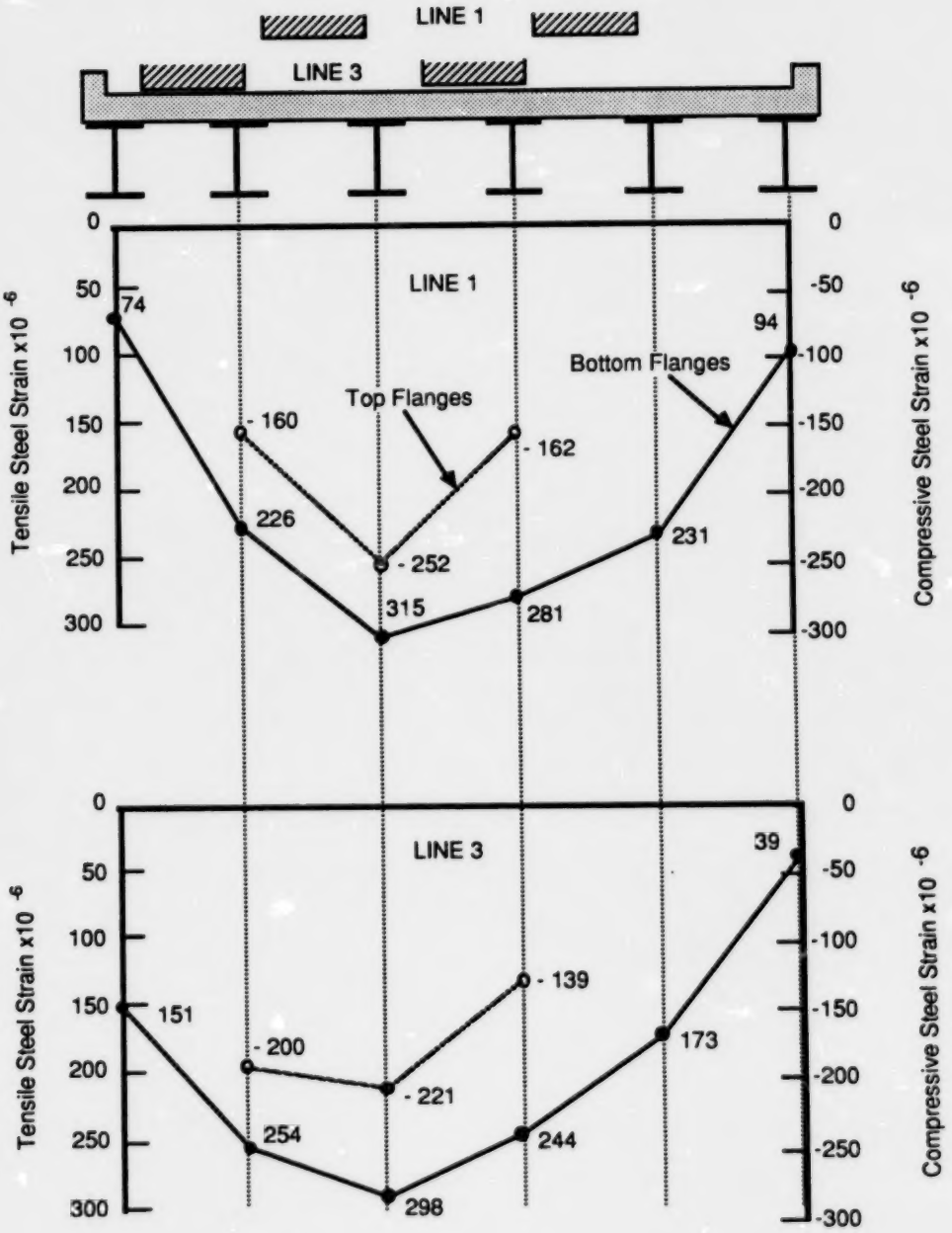


Figure 18/ Test ResultsSteel Stringer Strains, Span No.1, Load Level 6, Longitudinal Position 3

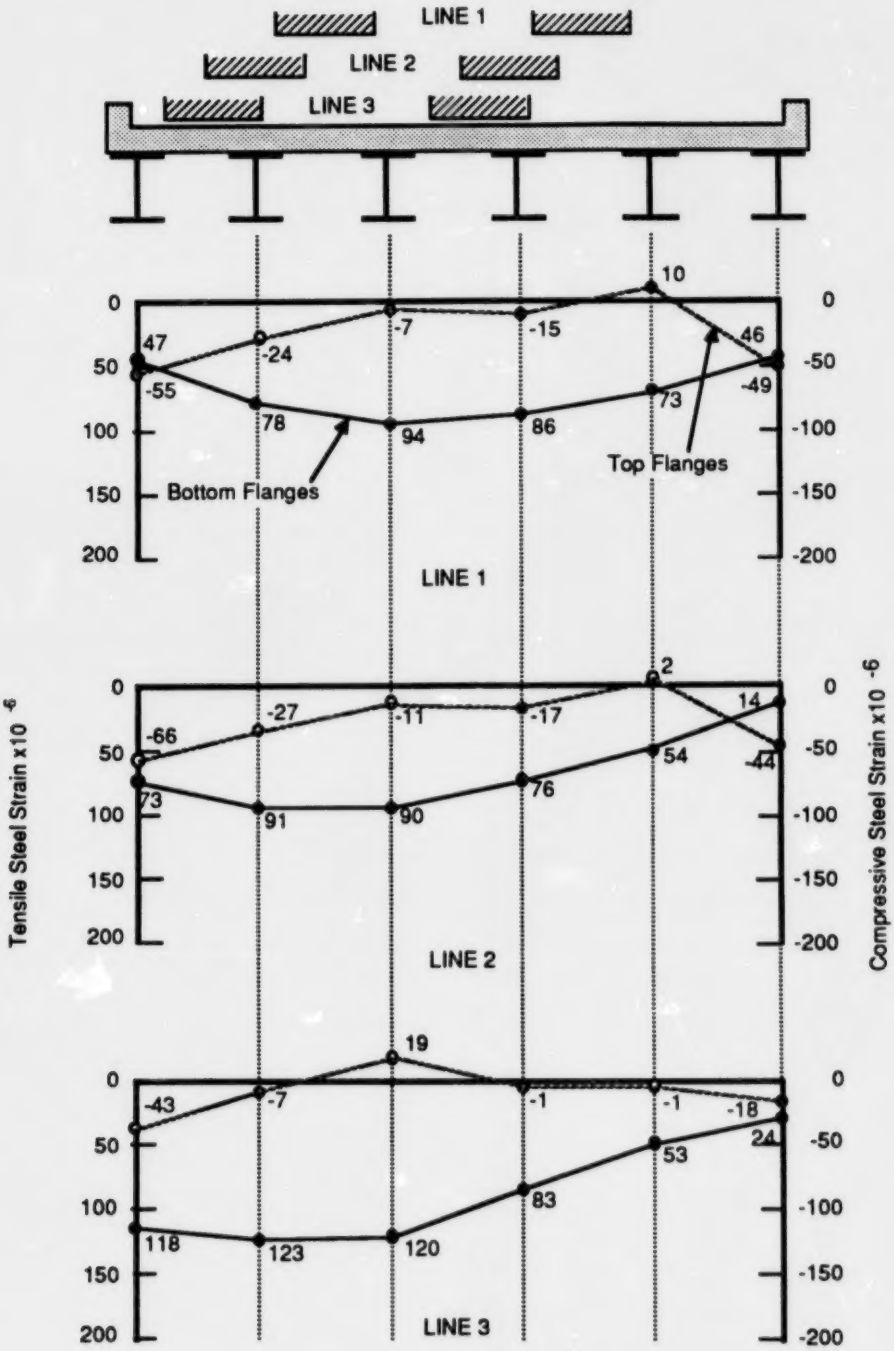


Figure 19/ Test Results–Steel Stringer Strains, Span No. 2, Load Level 1, Longitudinal Position 1

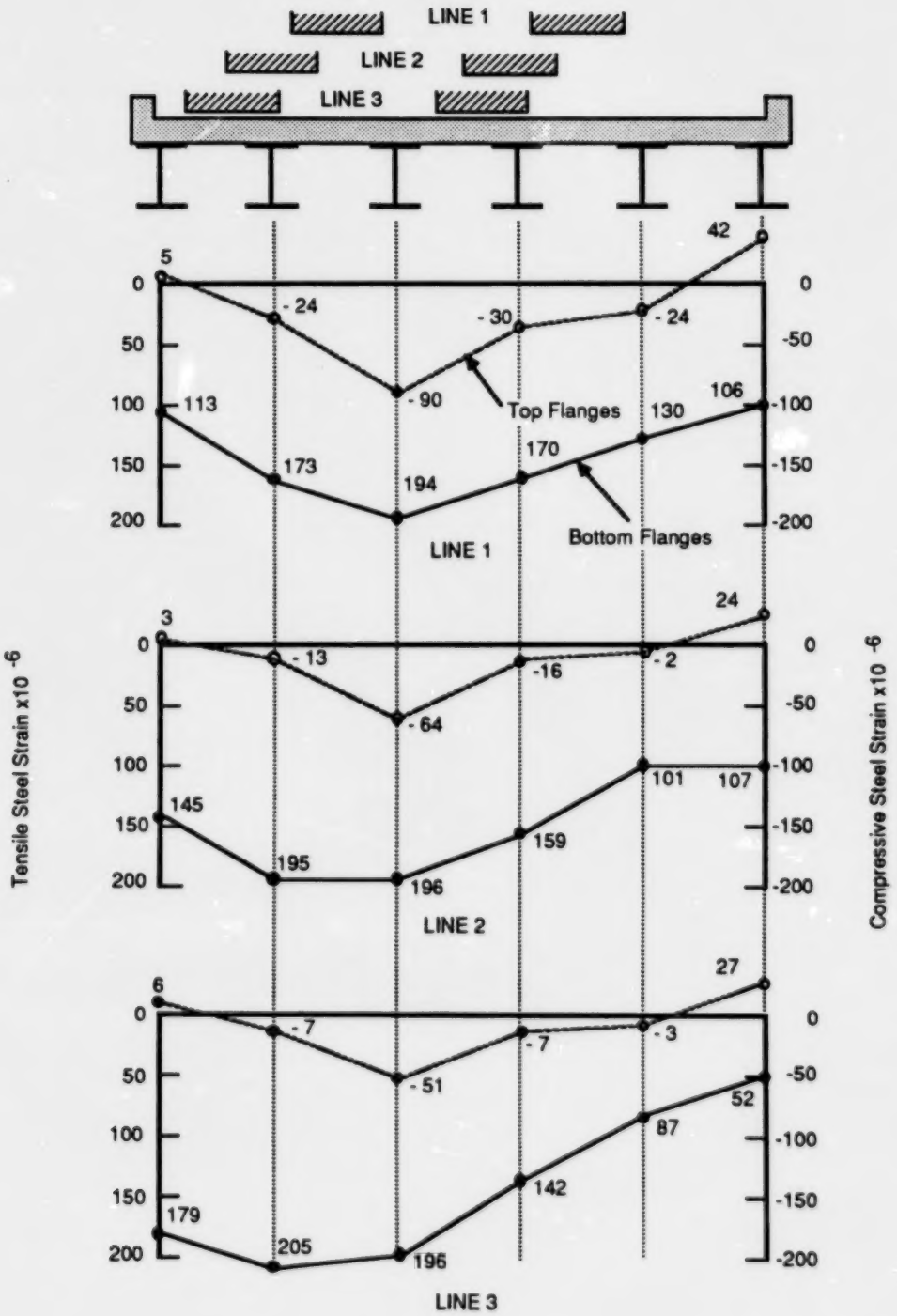


Figure 20/ Test Results–Steel Stringer Strains, Span No. 2, Load Level 3, Longitudinal Position 1

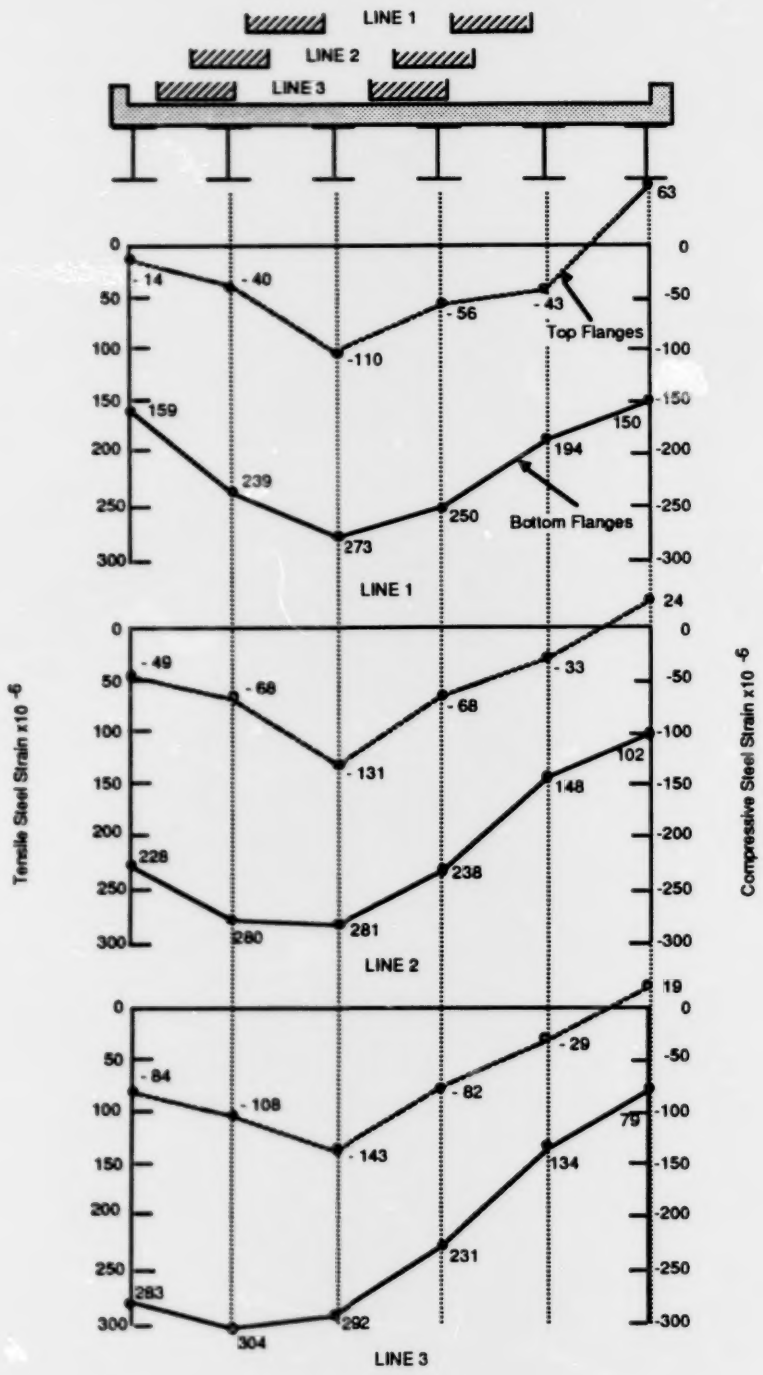


Figure 21/ Test Results–Steel Stringer Strains, Span No. 2, Load Level 5, Longitudinal Position 1

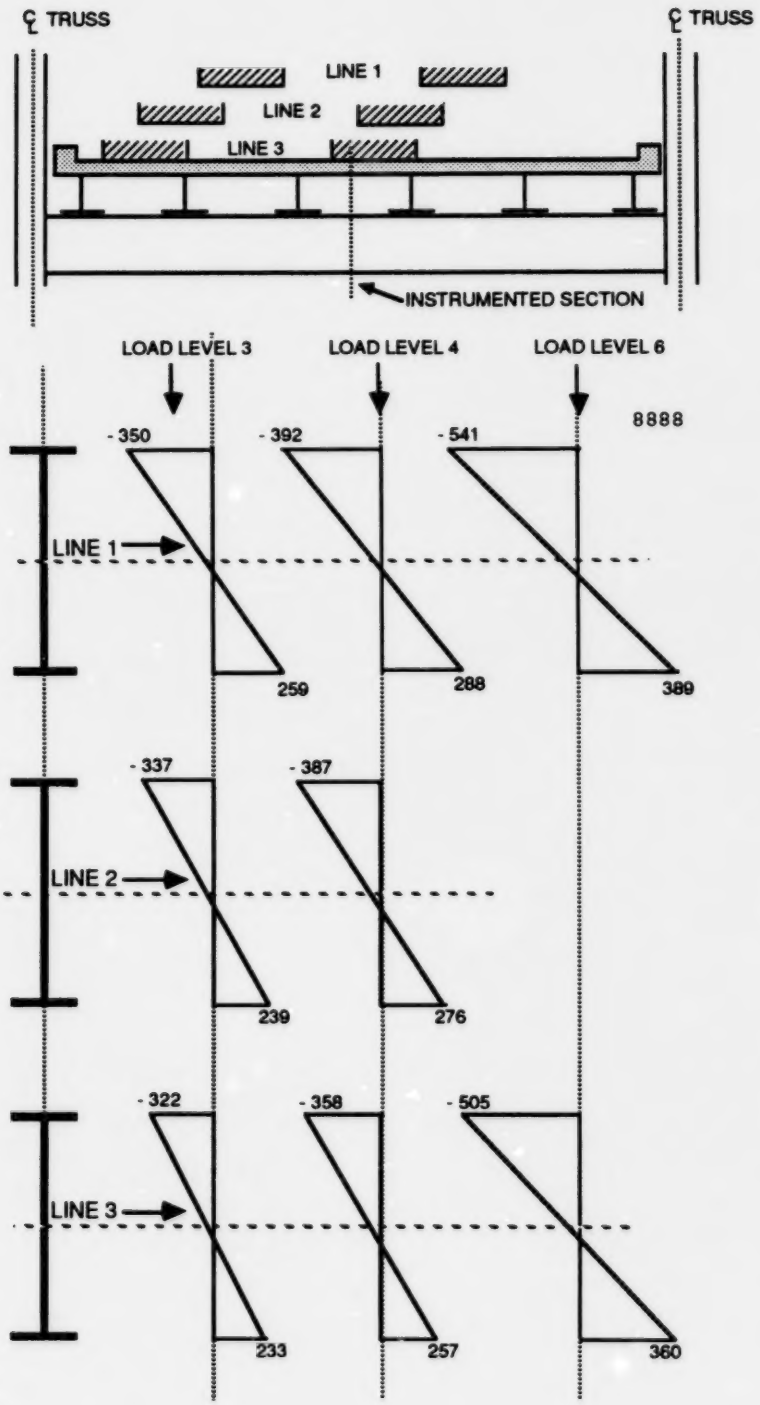


Figure 22/ Test Results–Steel Floor Beam Strains ($x10^{-6}$), Span No. 1, Floor Beam No. 3, Position 2

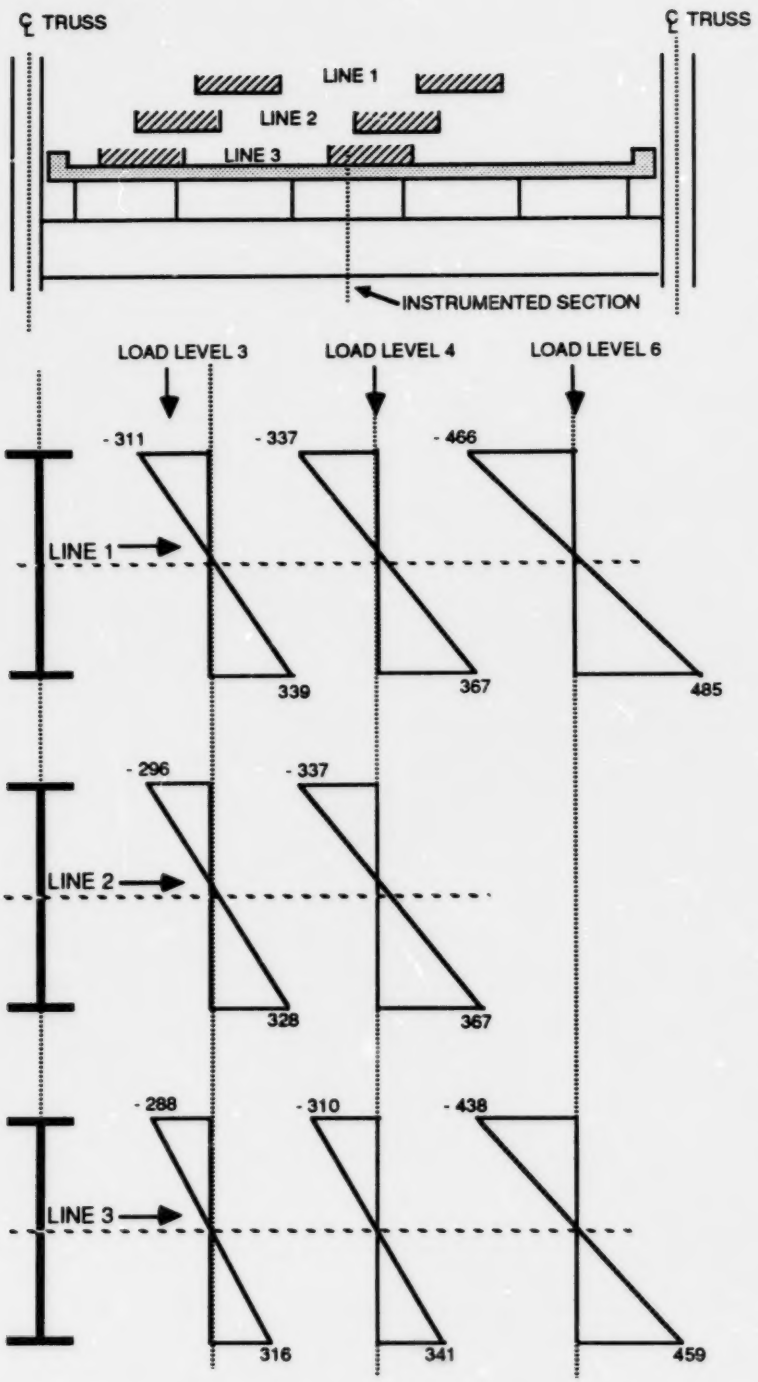


Figure 23/ Test Results–Steel Floor Beam Strains ($x10^{-6}$), Span No. 1, Floor Beam No. 4 Position 6

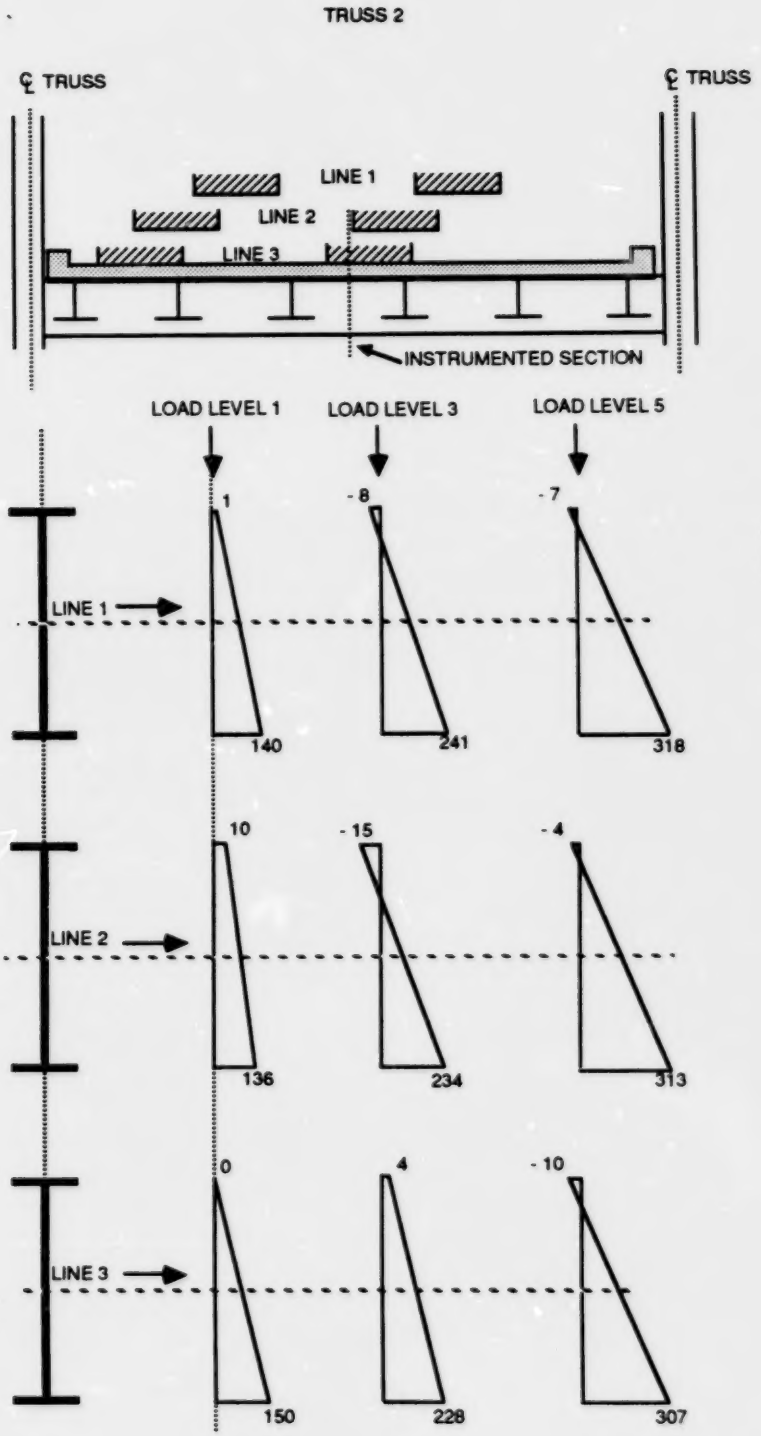


Figure 24/ Test Results–Steel Floor Beam Strains ($x10^{-6}$), Span No. 2, Floor Beam No. 2, Position 3

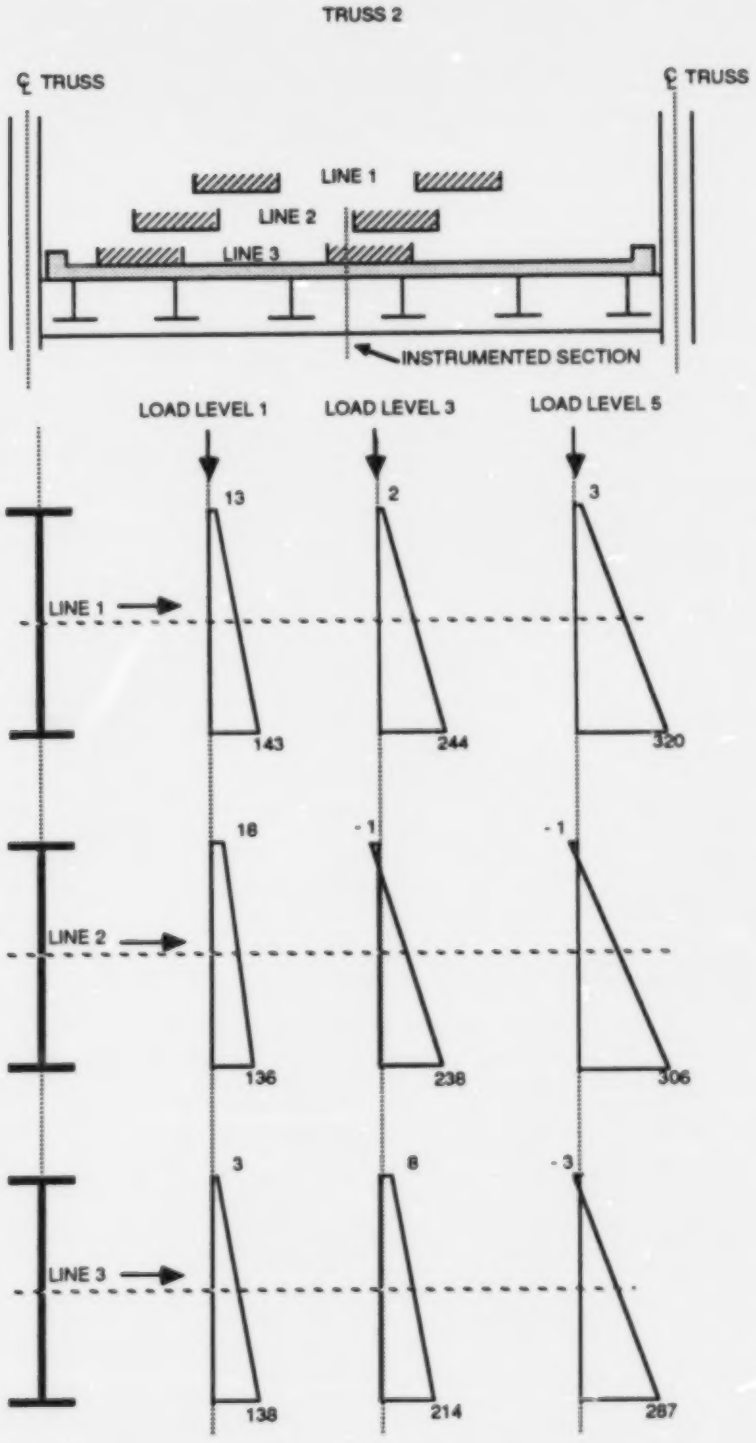


Figure 25/ Test Results–Steel Floor Beam Strains ($x10^{-6}$), Span No. 2, Floor Beam No. 3, Position 2

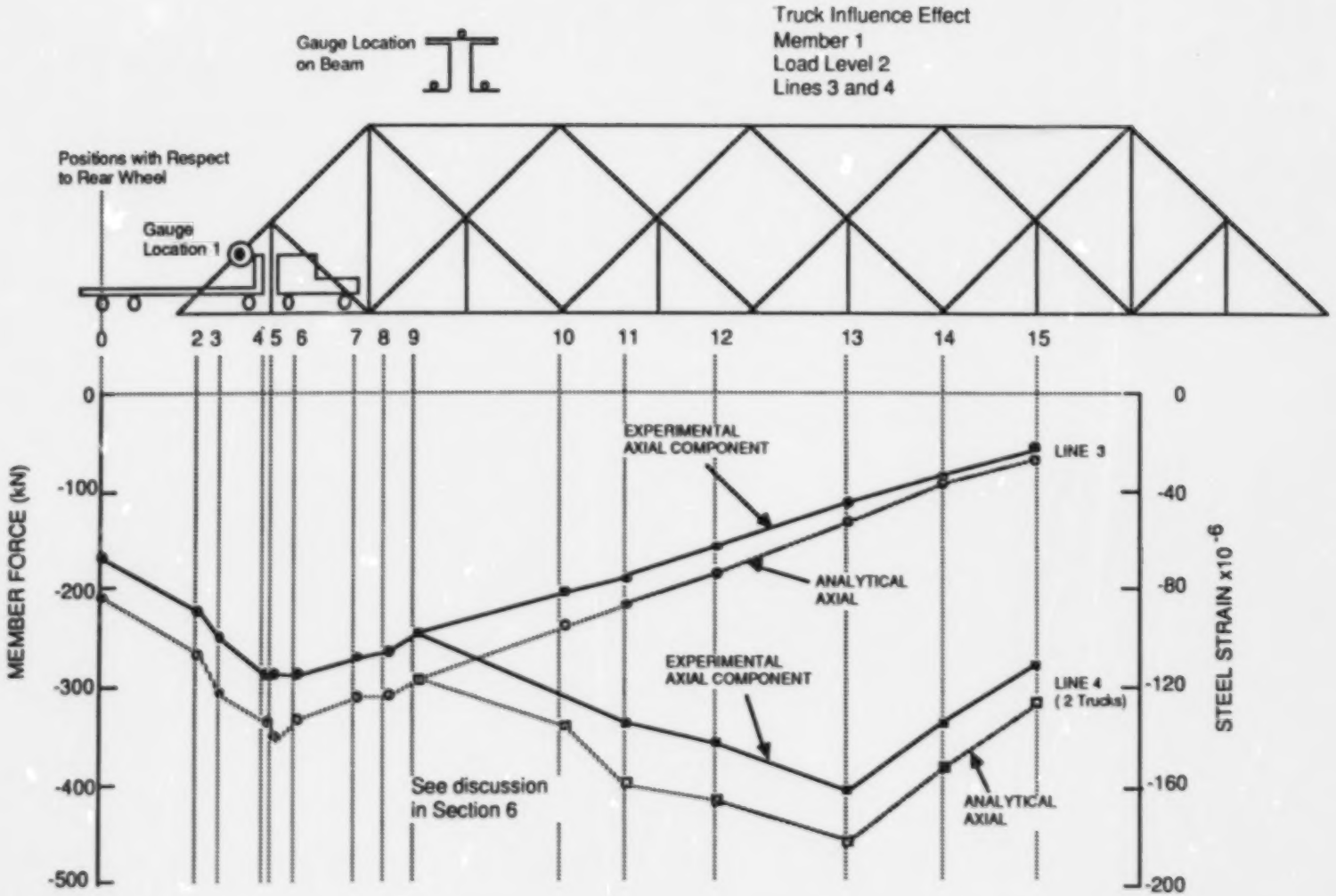


Figure 26/ Member Forces and Strains–Steel Truss, Span No. 1, Member 1, Load Level 2

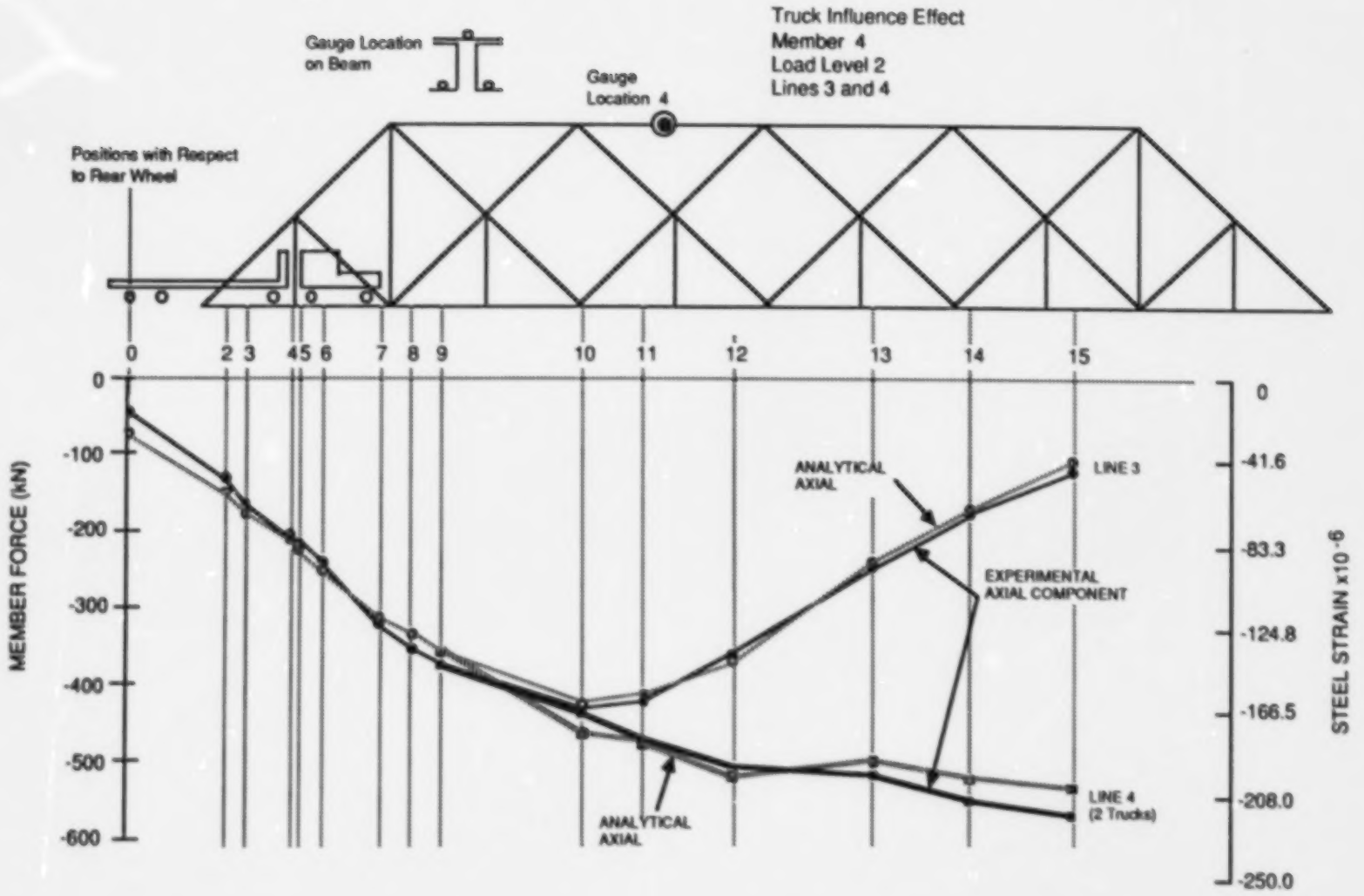


Figure 27/ Member Forces and Strains–Steel Truss, Span No. 1, Member 4, Load Level 2

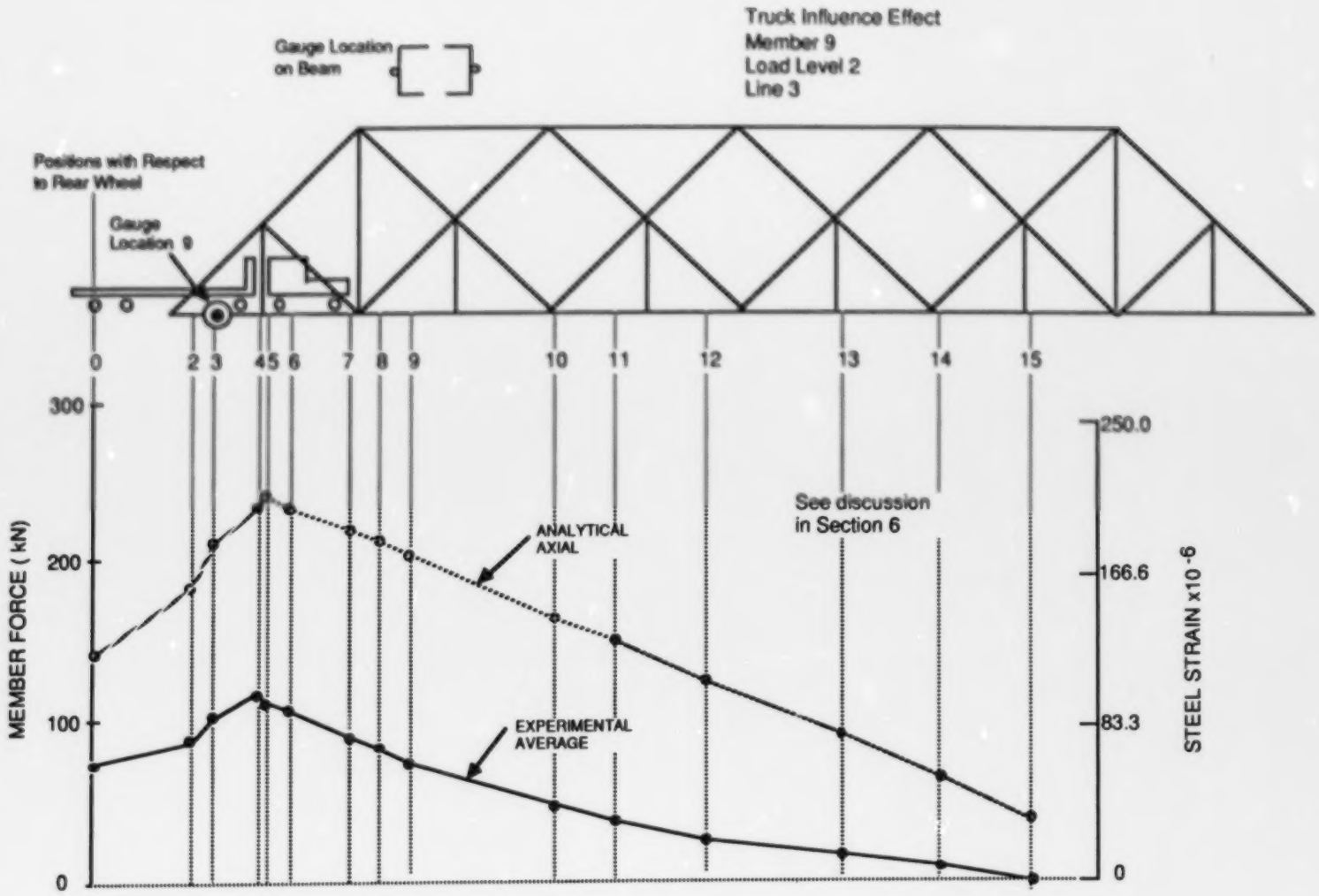


Figure 28/ Member Forces and Strains–Steel Truss, Span No. 1, Member 9, Load Level 2

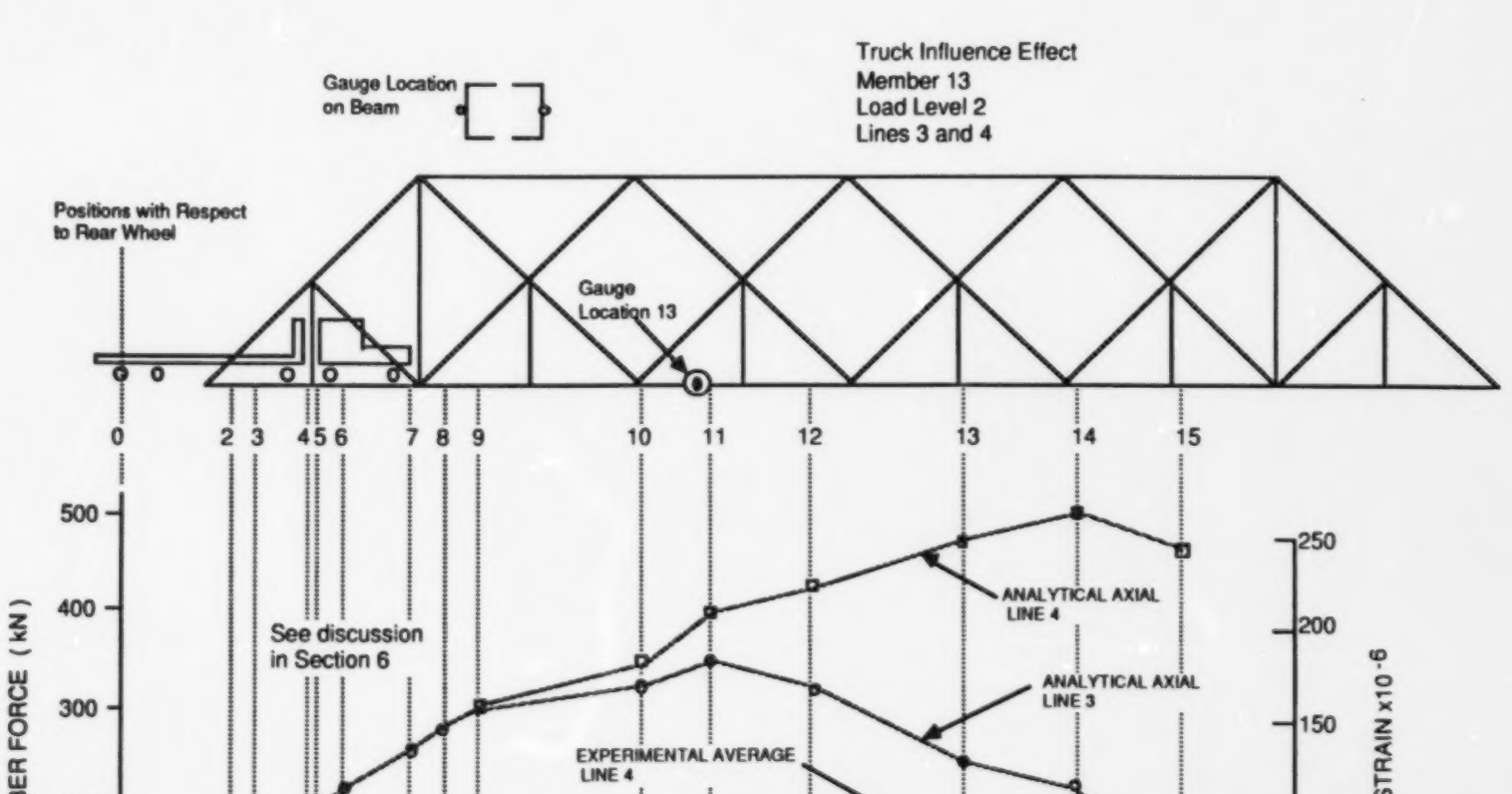


Figure 29/ Member Forces and Strains–Steel Truss, Span No. 1, Member 13, Load Level 2

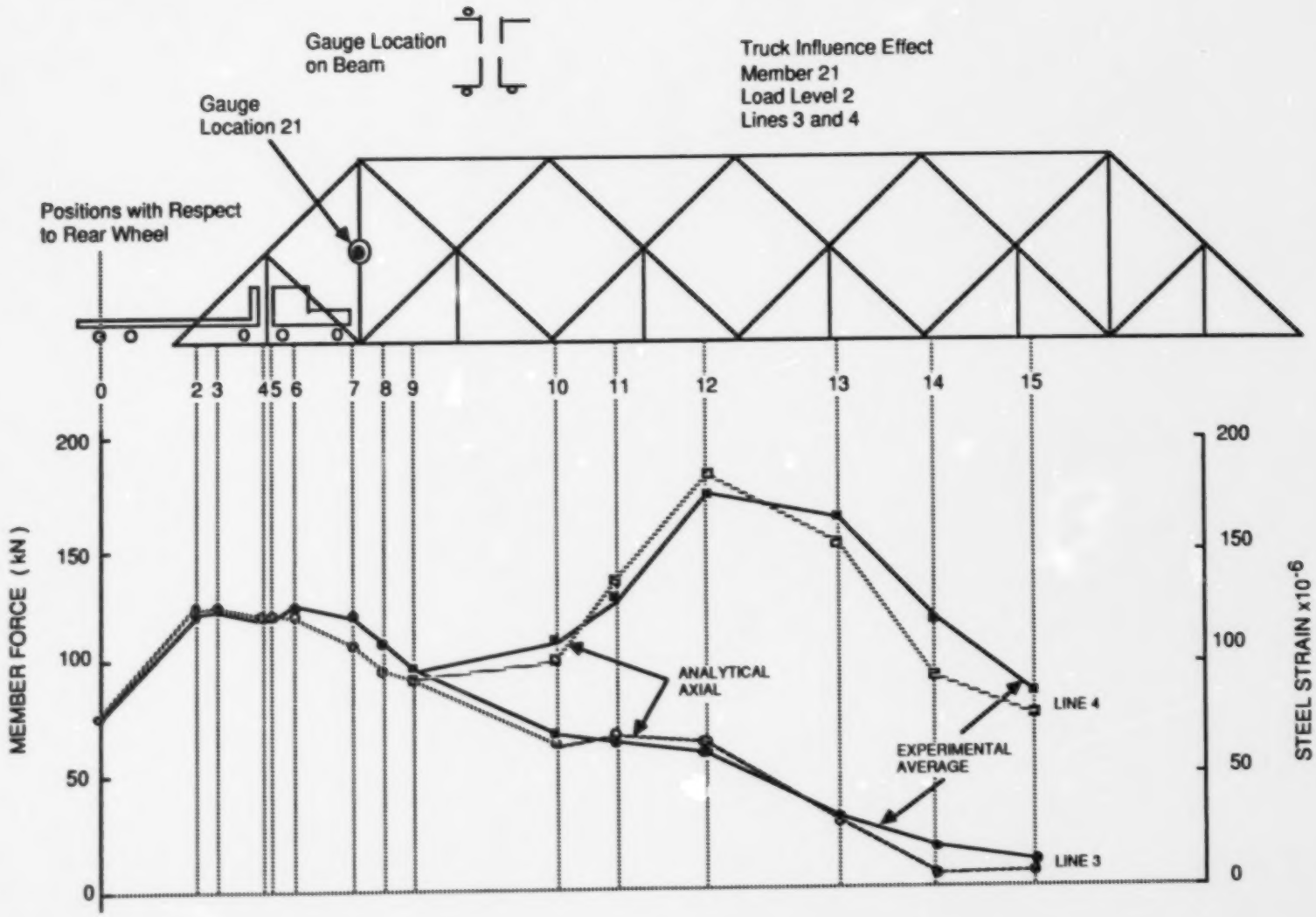


Figure 30/ Member Forces and Strains–Steel Truss, Span No. 1, Member 21, Load Level 2

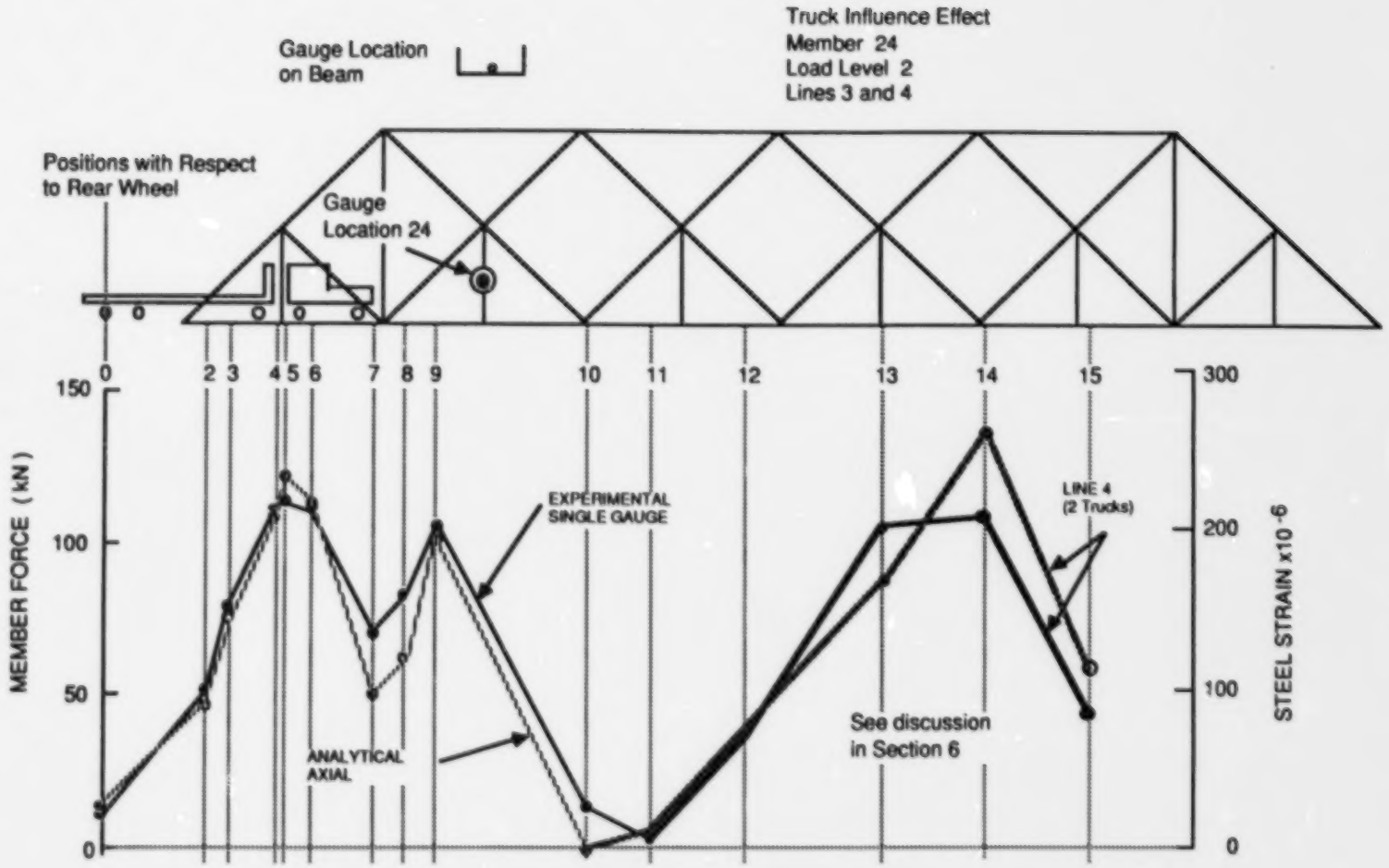


Figure 31/ Member Forces and Strains–Steel Truss, Span No. 1, Member 24, Load Level 2

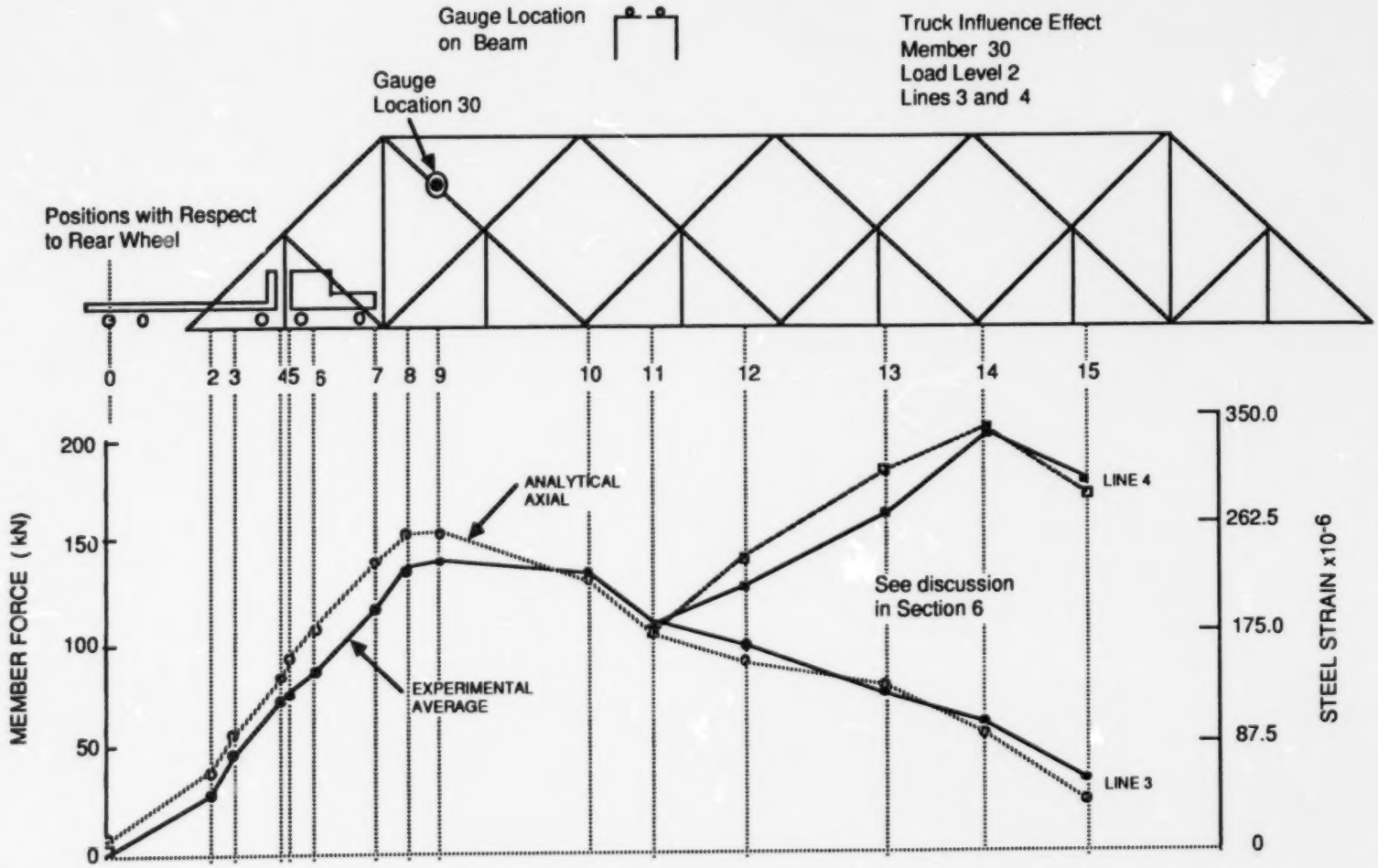


Figure 32/ Member Forces and Strains–Steel Truss, Span No. 1, Member 30, Load Level 2

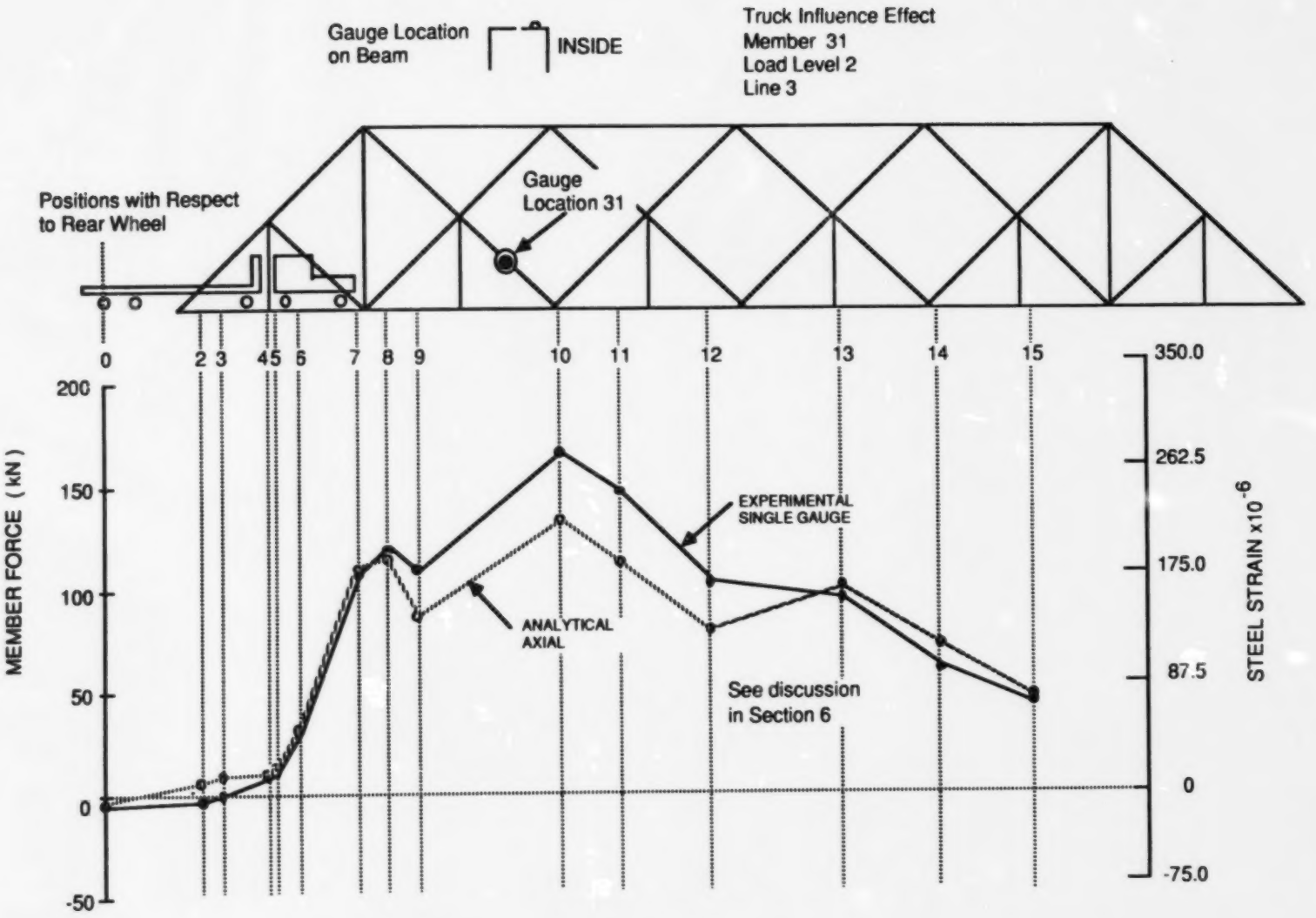


Figure 33/ Member Forces and Strains–Steel Truss, Span No. 1, Member 31. Load Level 23

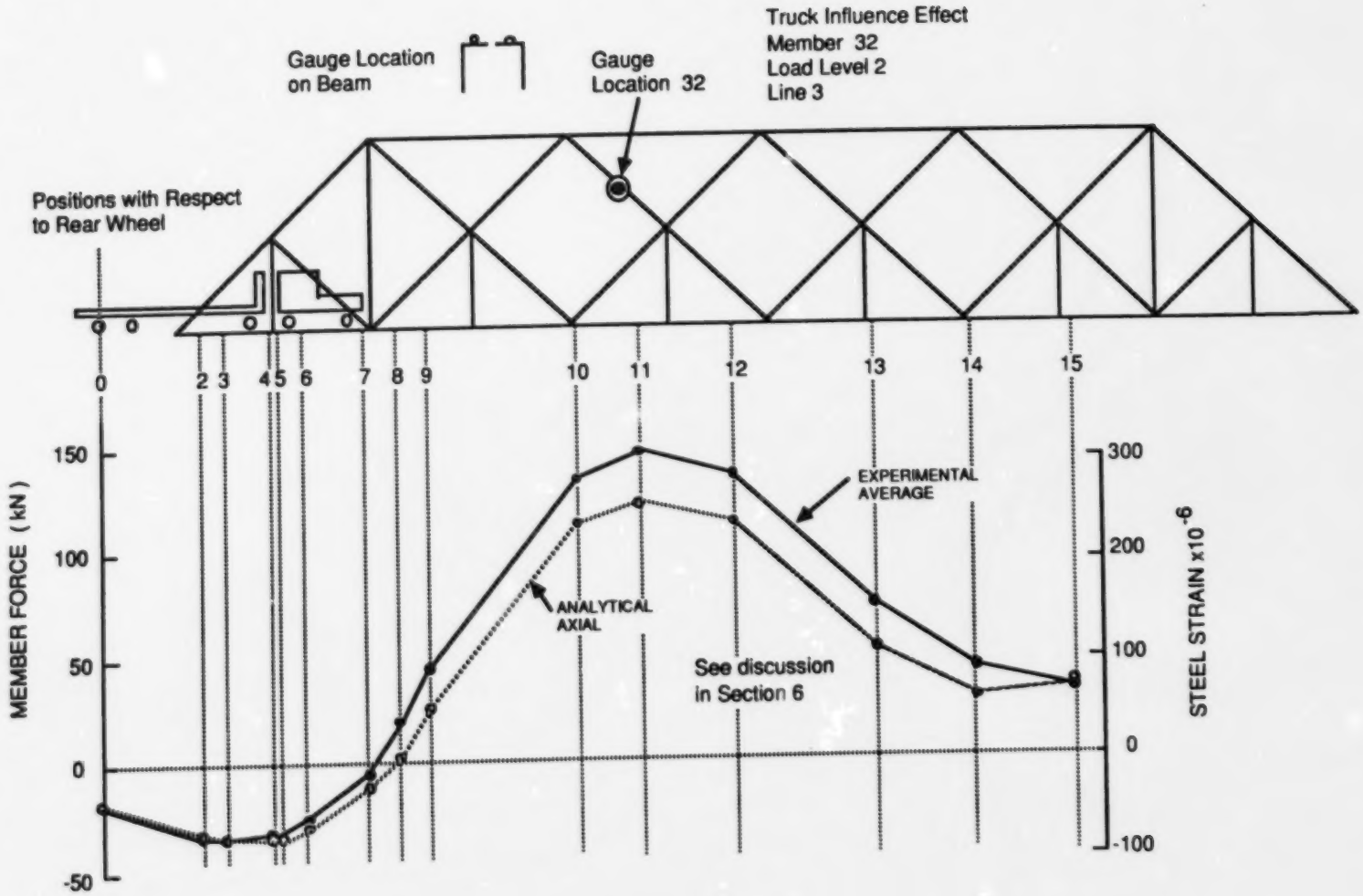


Figure 34/ Member Forces and Strains–Steel Truss, Span No. 1, Member 32, Load Level 2

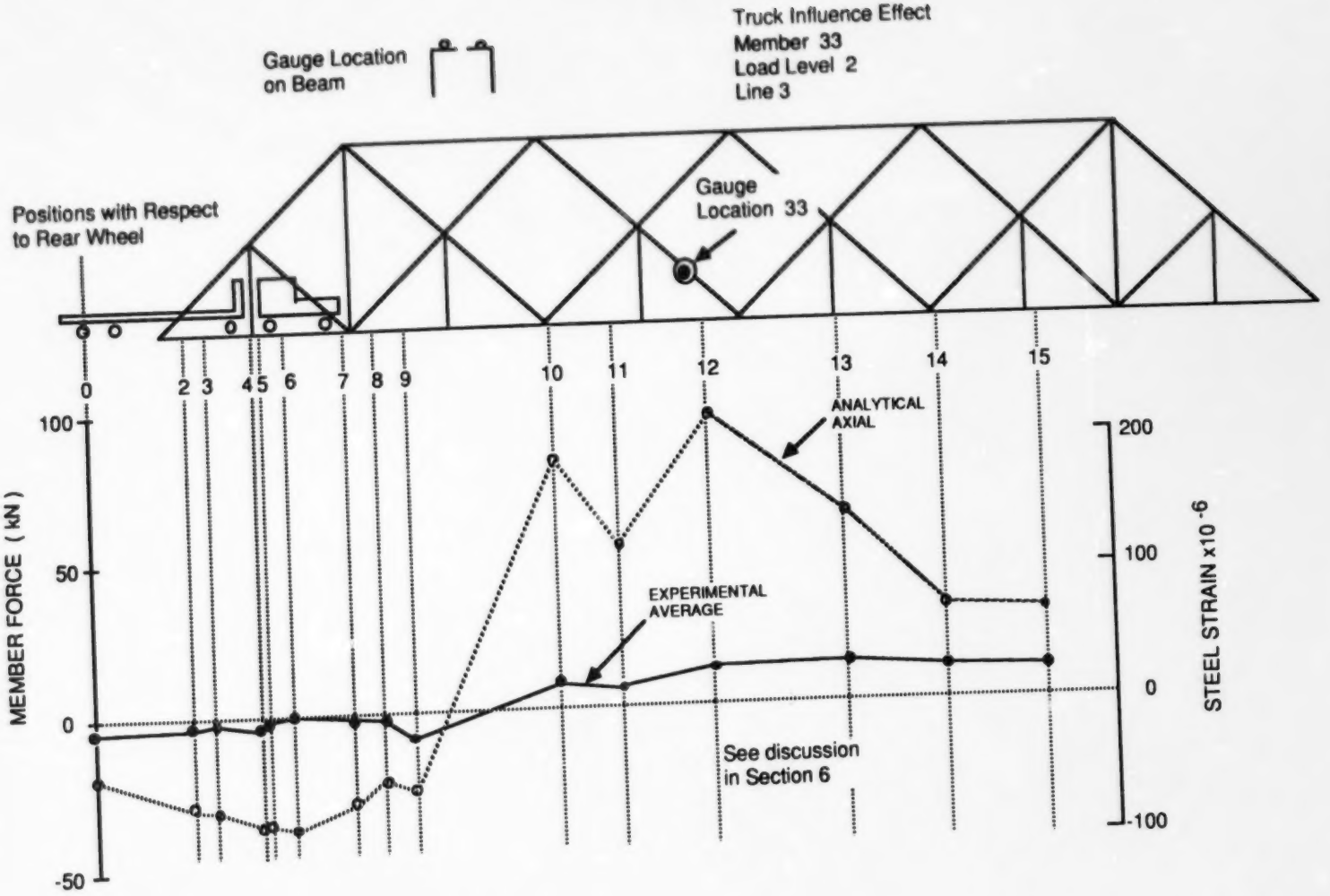


Figure 35/ Member Forces and Strains–Steel Truss, Span No. 1, Member 33, Load Level 2

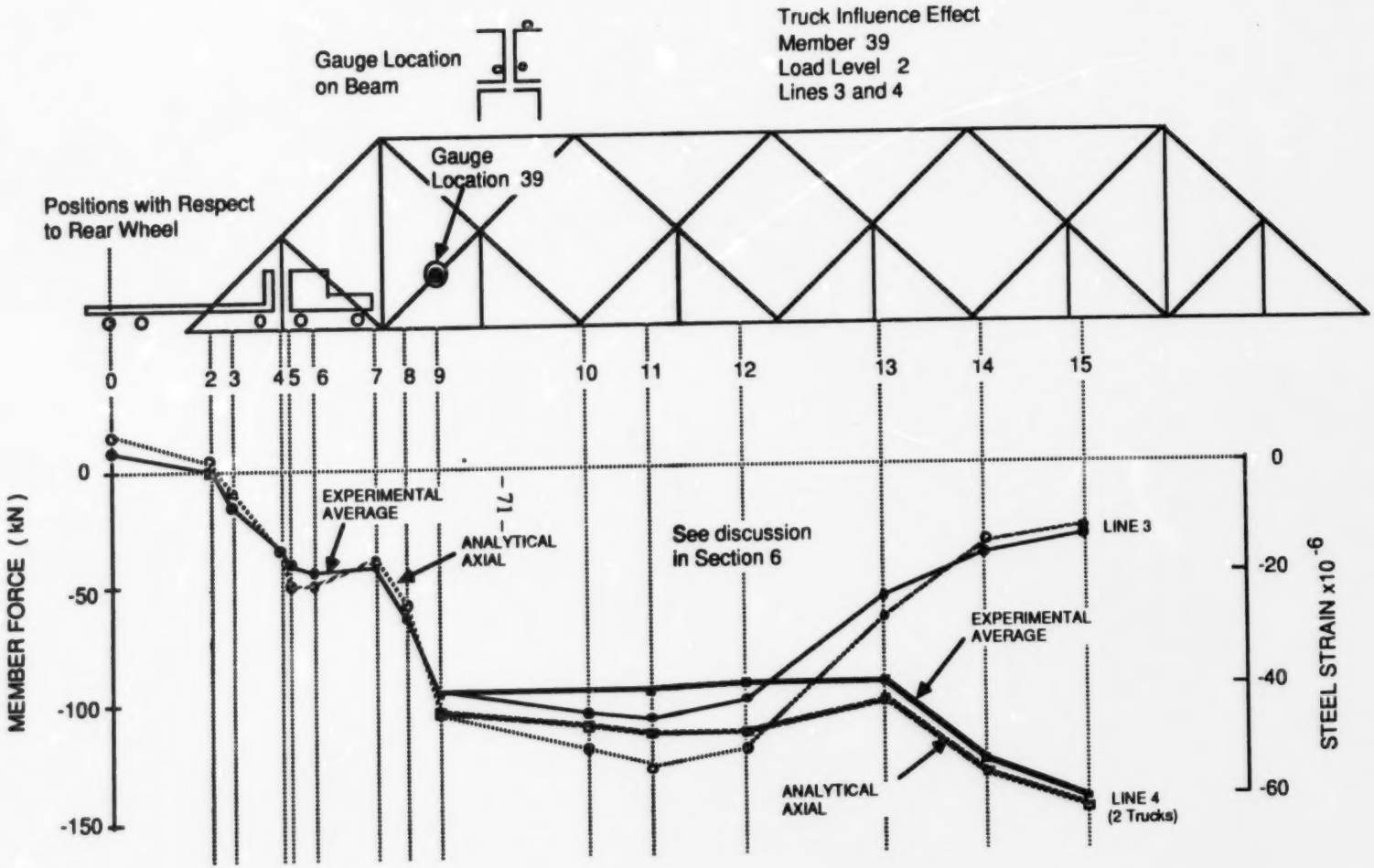


Figure 36/ Member Forces and Strains–Steel Truss, Span No. 1, Member 39, Load Level 2

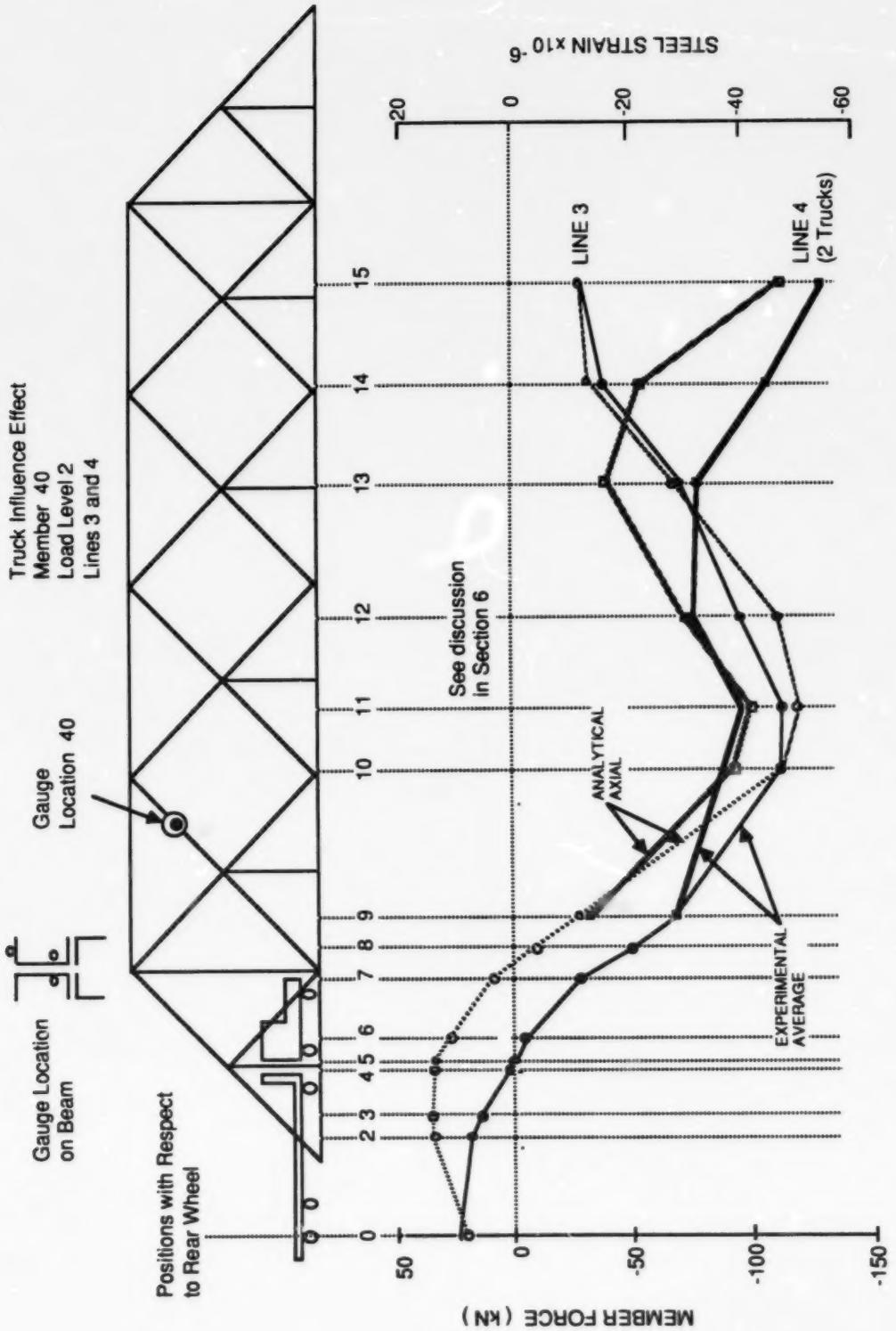


Figure 37/ Member Forces and Strains–Steel Truss, Span No. 1, Member 40, Load Level 2

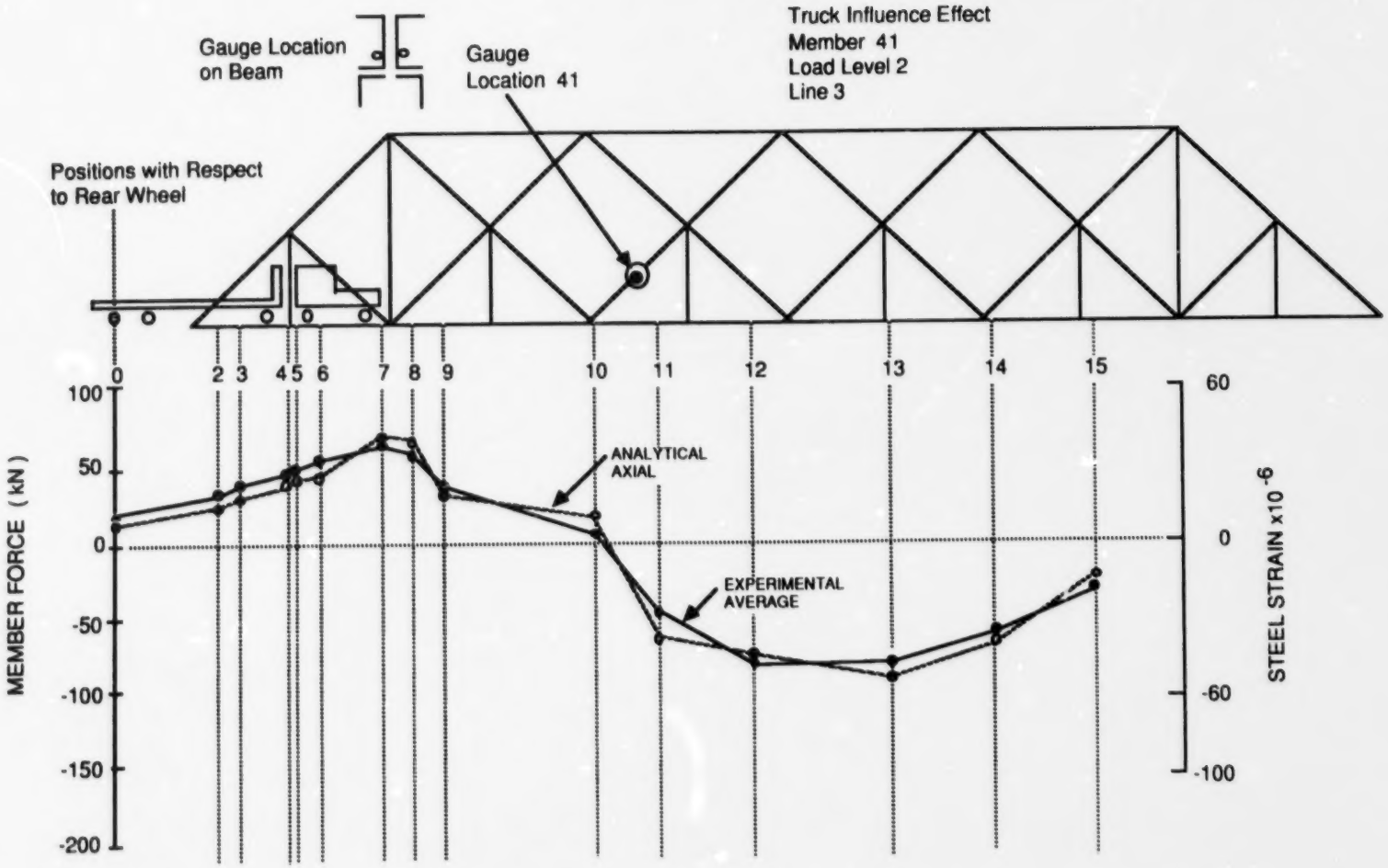


Figure 38/ Member Forces and Strains–Steel Truss, Span No. 1, Member 41, Load Level 2

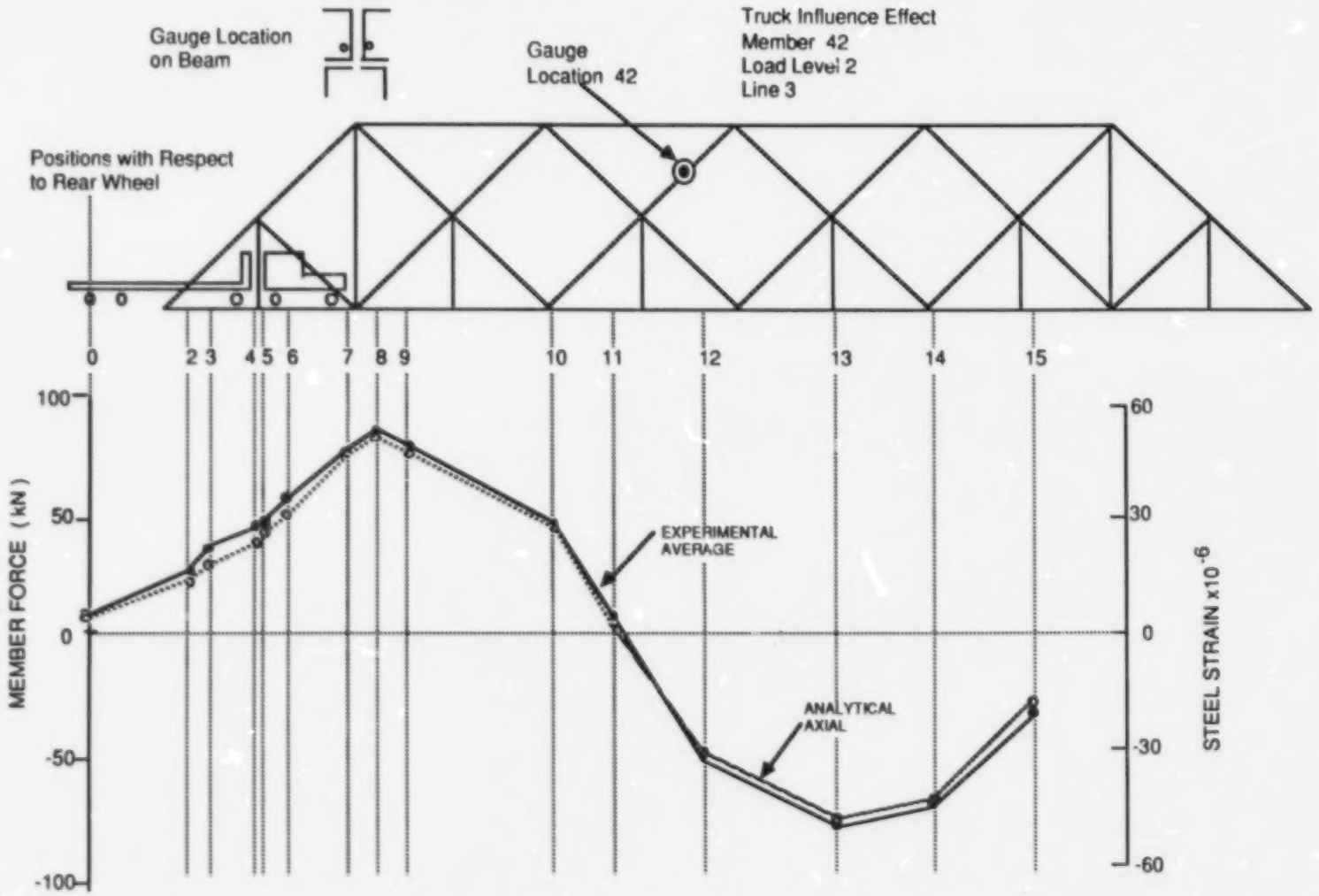


Figure 39/ Member Forces and Strains–Steel Truss, Span No. 1, Member 42, Load Level 2

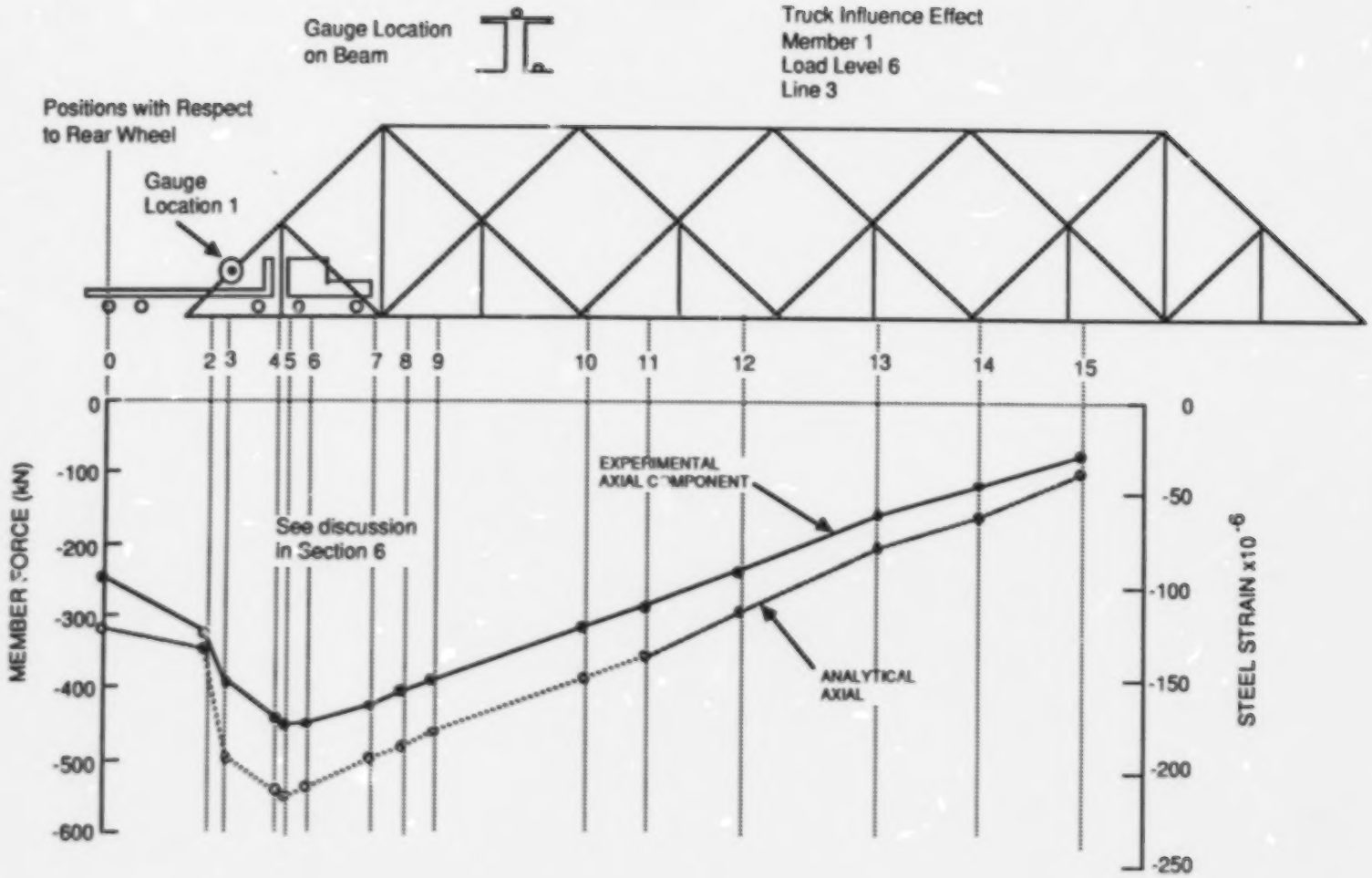


Figure 40/ Member Forces and Strains–Steel Truss, Span No. 1, Member 1, Load Level 6

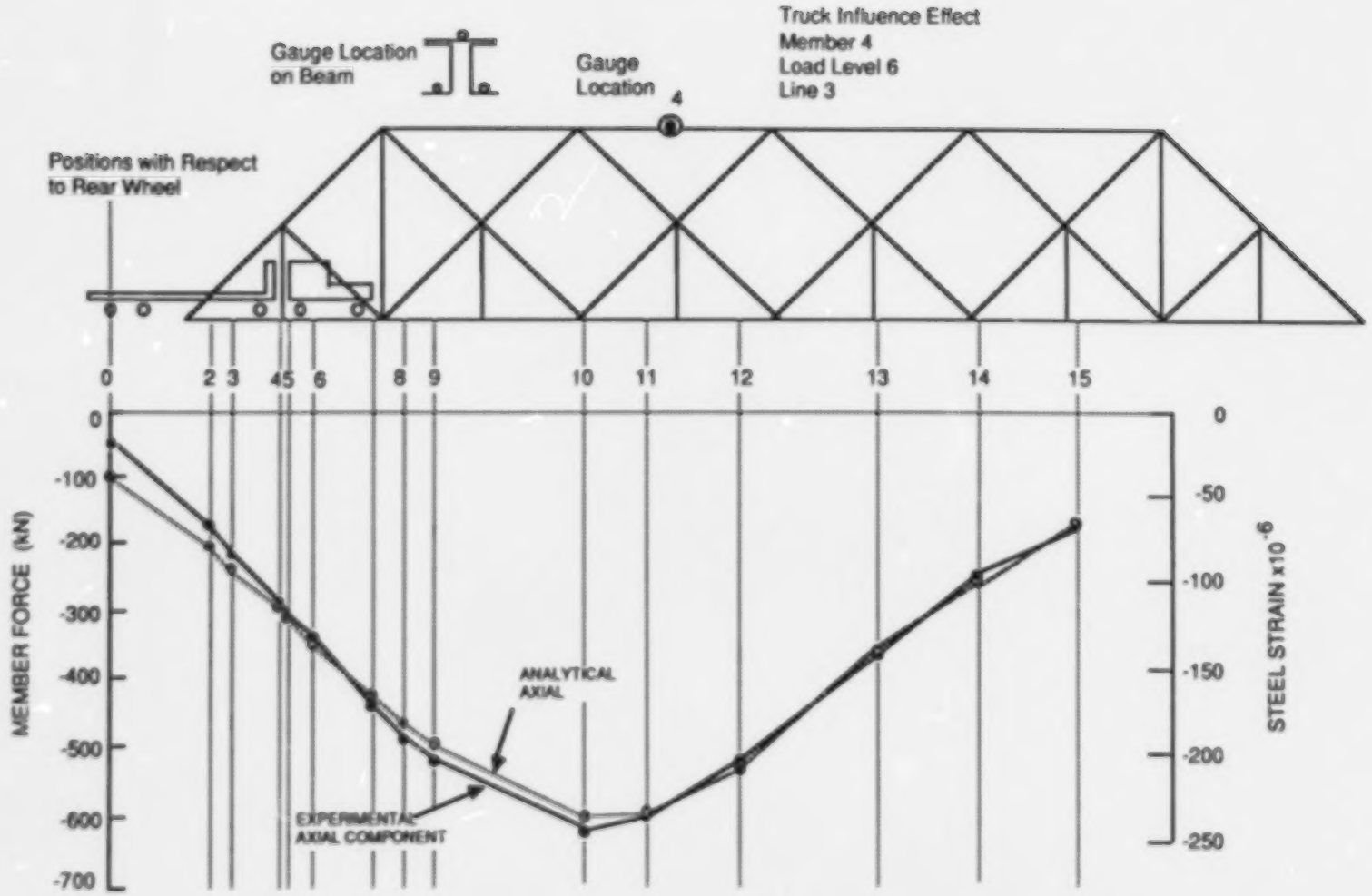


Figure 41/ Member Forces and Strains–Steel Truss, Span No. 1, Member 4, Load Level 6

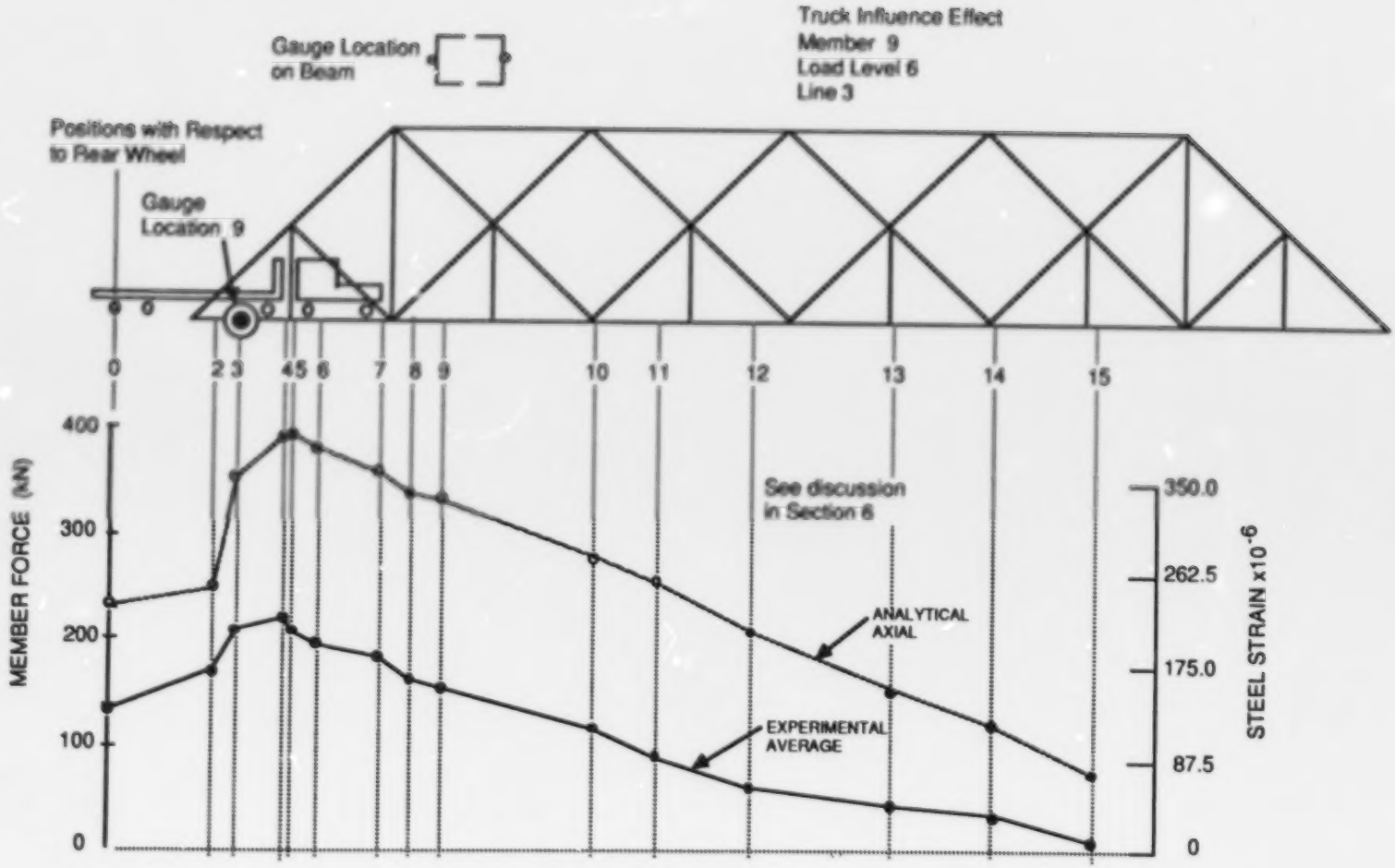


Figure 42/ Member Forces and Strains–Steel Truss, Span No. 1, Member 9, Load Level 6

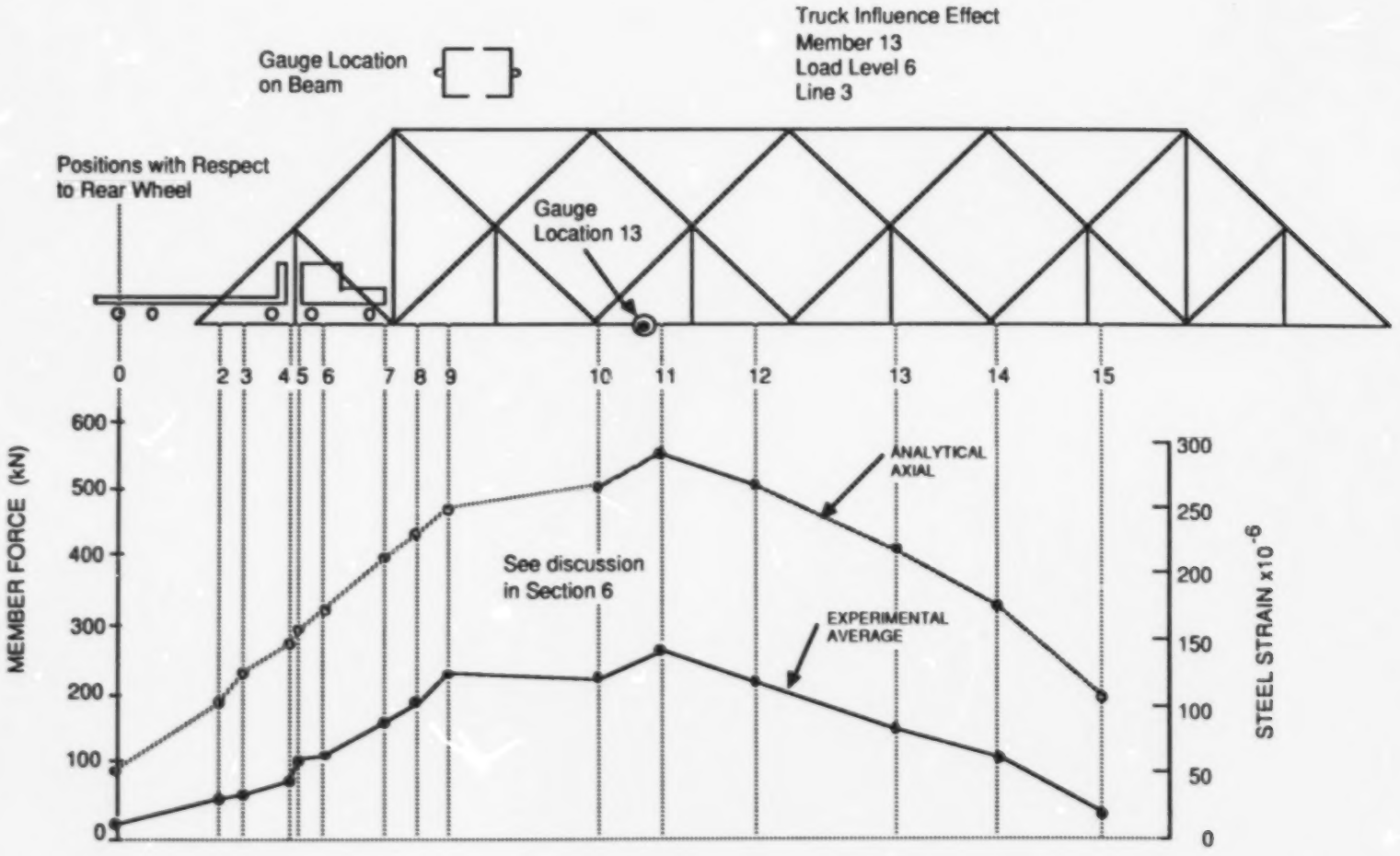


Figure 43/ Member Forces and Strains–Steel Truss, Span No. 1, Member 13, Load Level 6

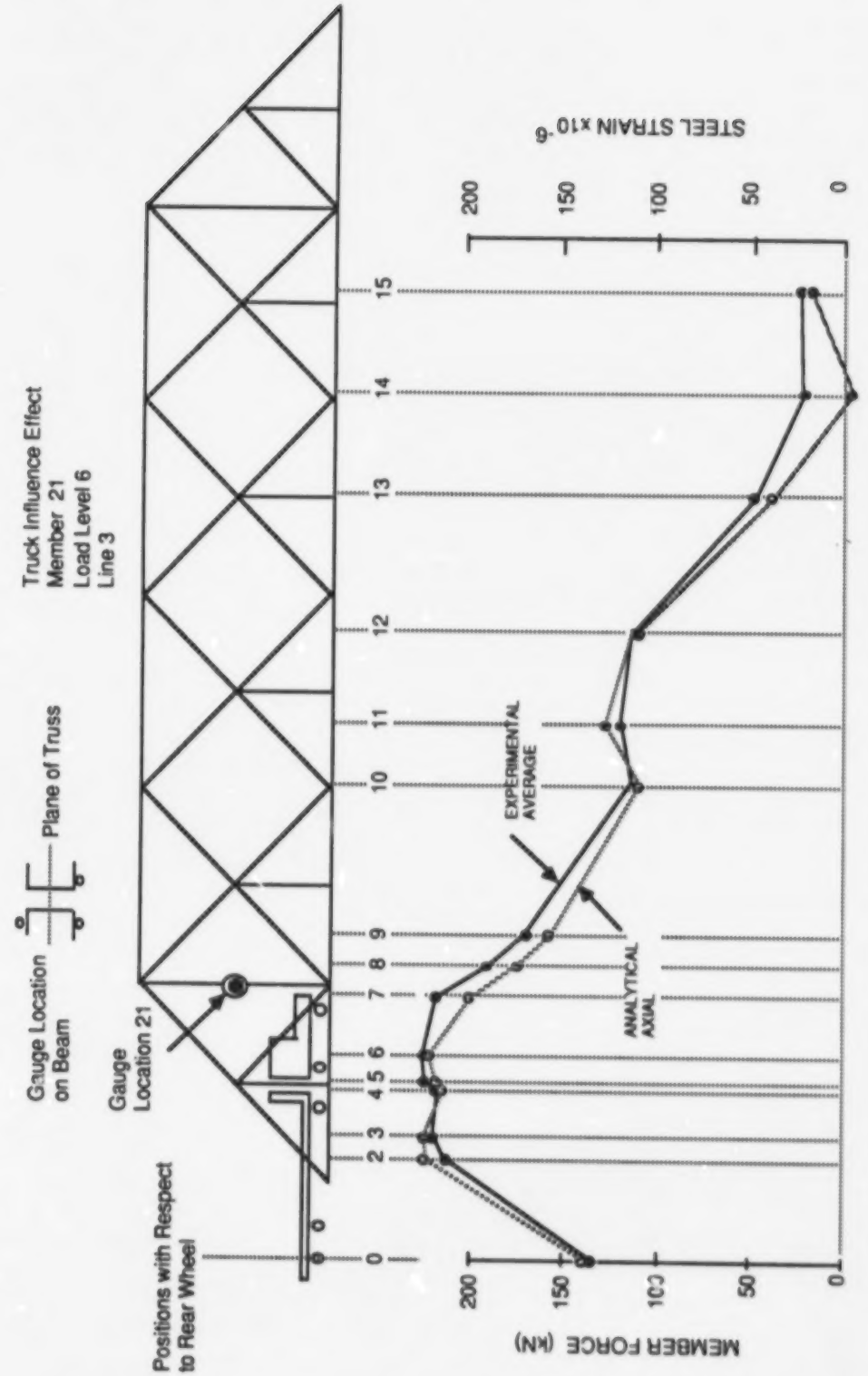


Figure 44/ Member Forces and Strains–Steel Truss, Span No. 1, Member 21, Load Level 6

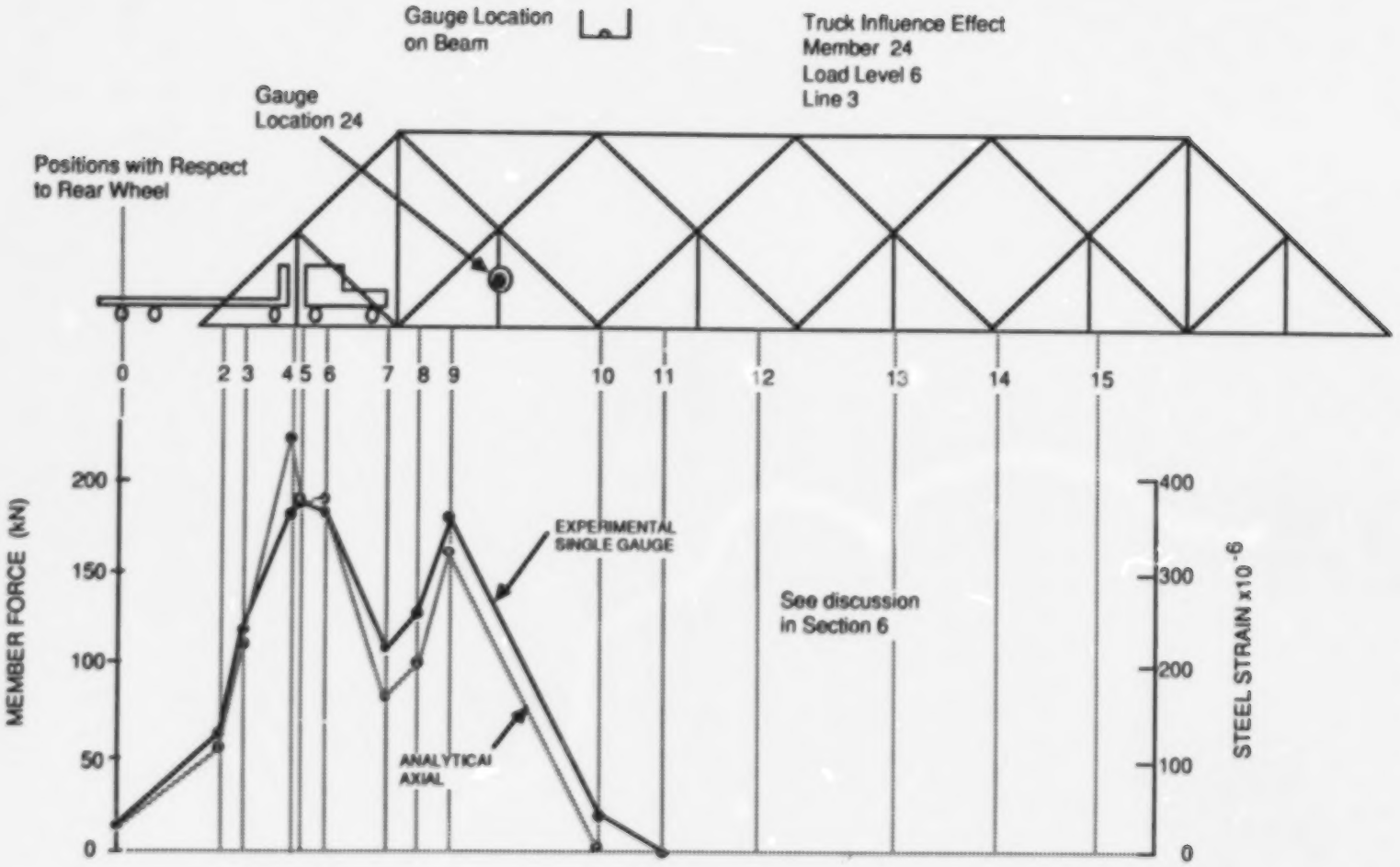


Figure 45/ Member Forces and Strains–Steel Truss, Span No. 1, Member 24, Load Level 6

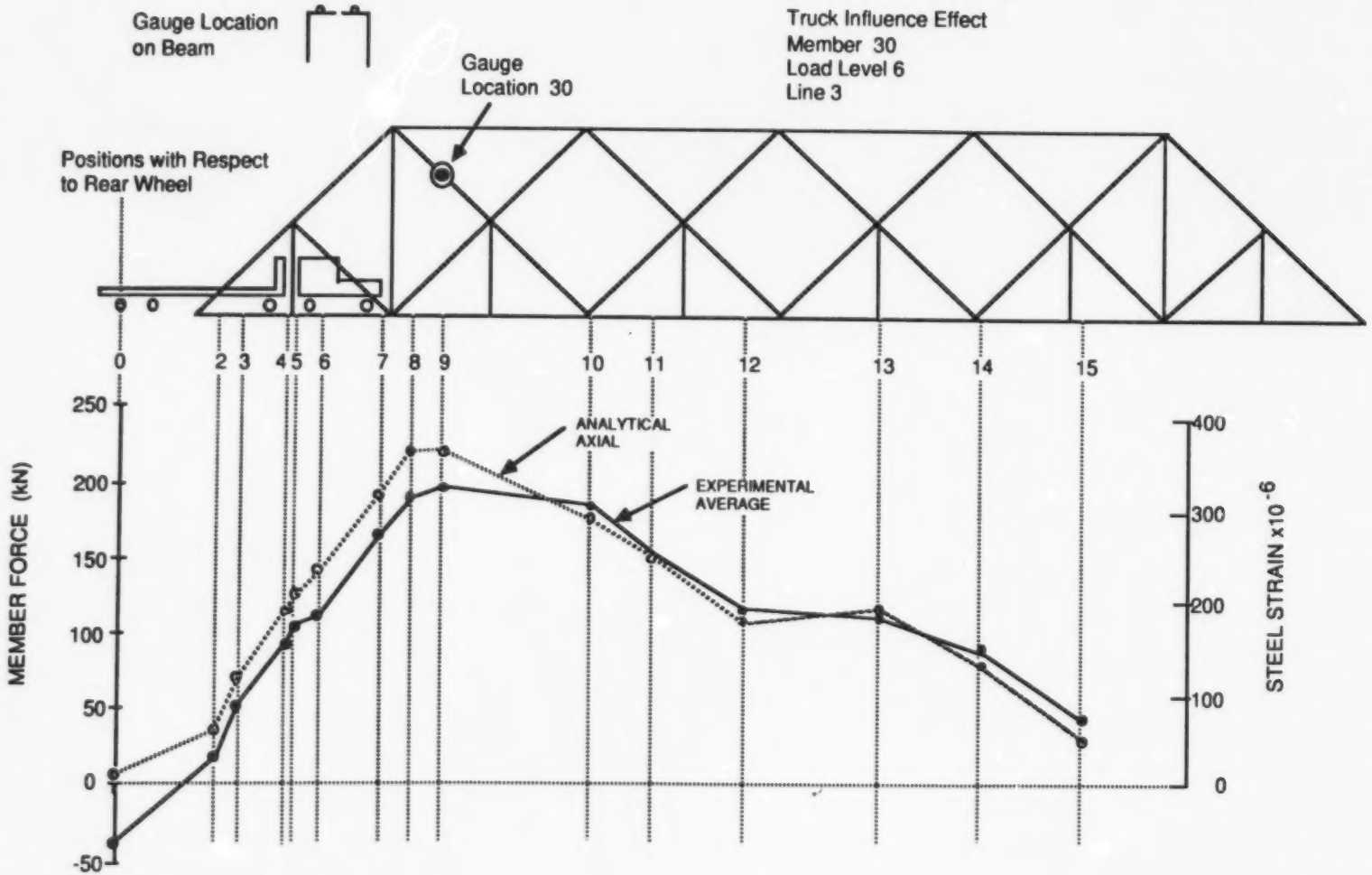


Figure 46/ Member Forces and Strains–Steel Truss, Span No. 1, Member 30, Load Level 6

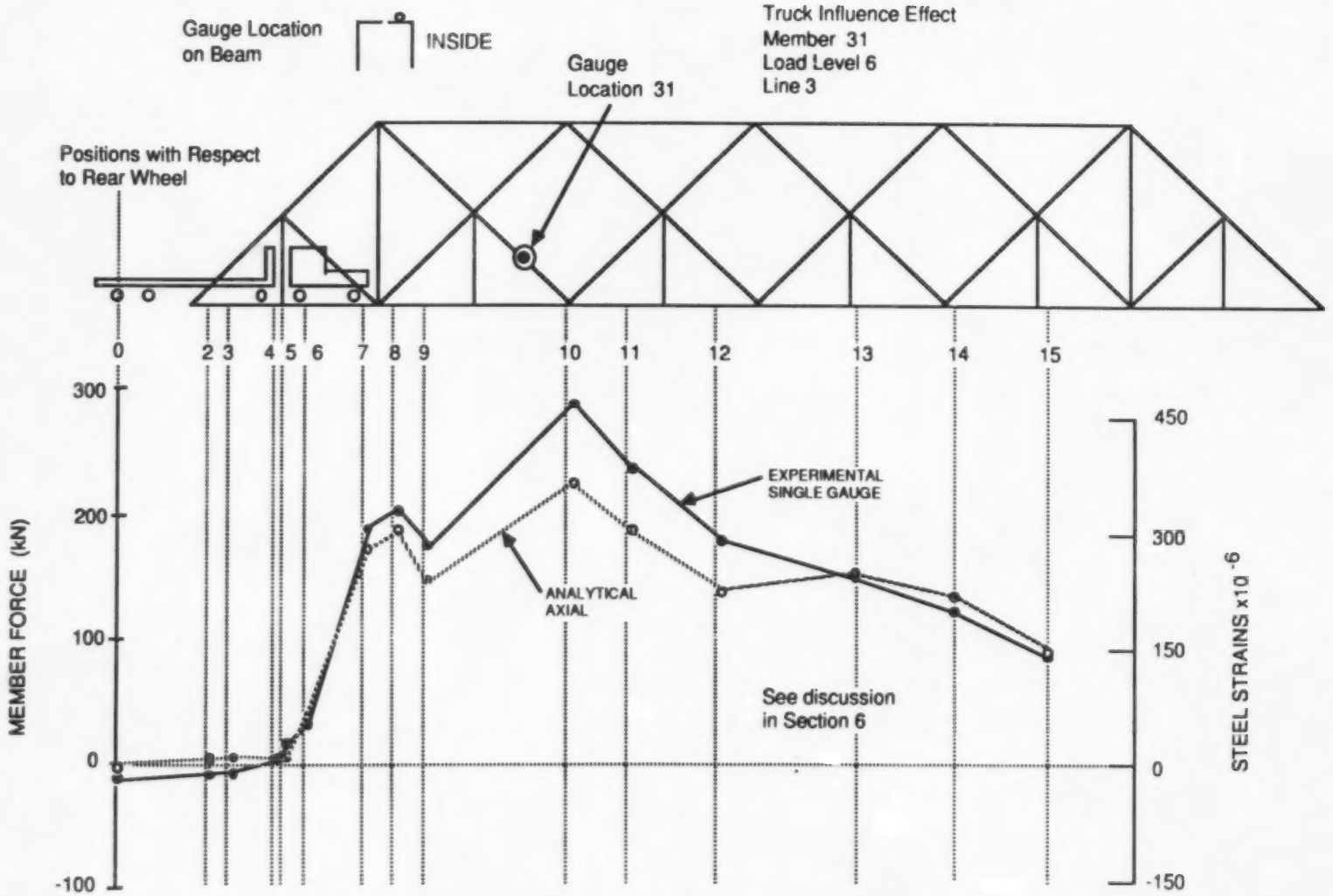


Figure 47/ Member Forces and Strains–Steel Truss, Span No. 1, Member 31, Load Level 6

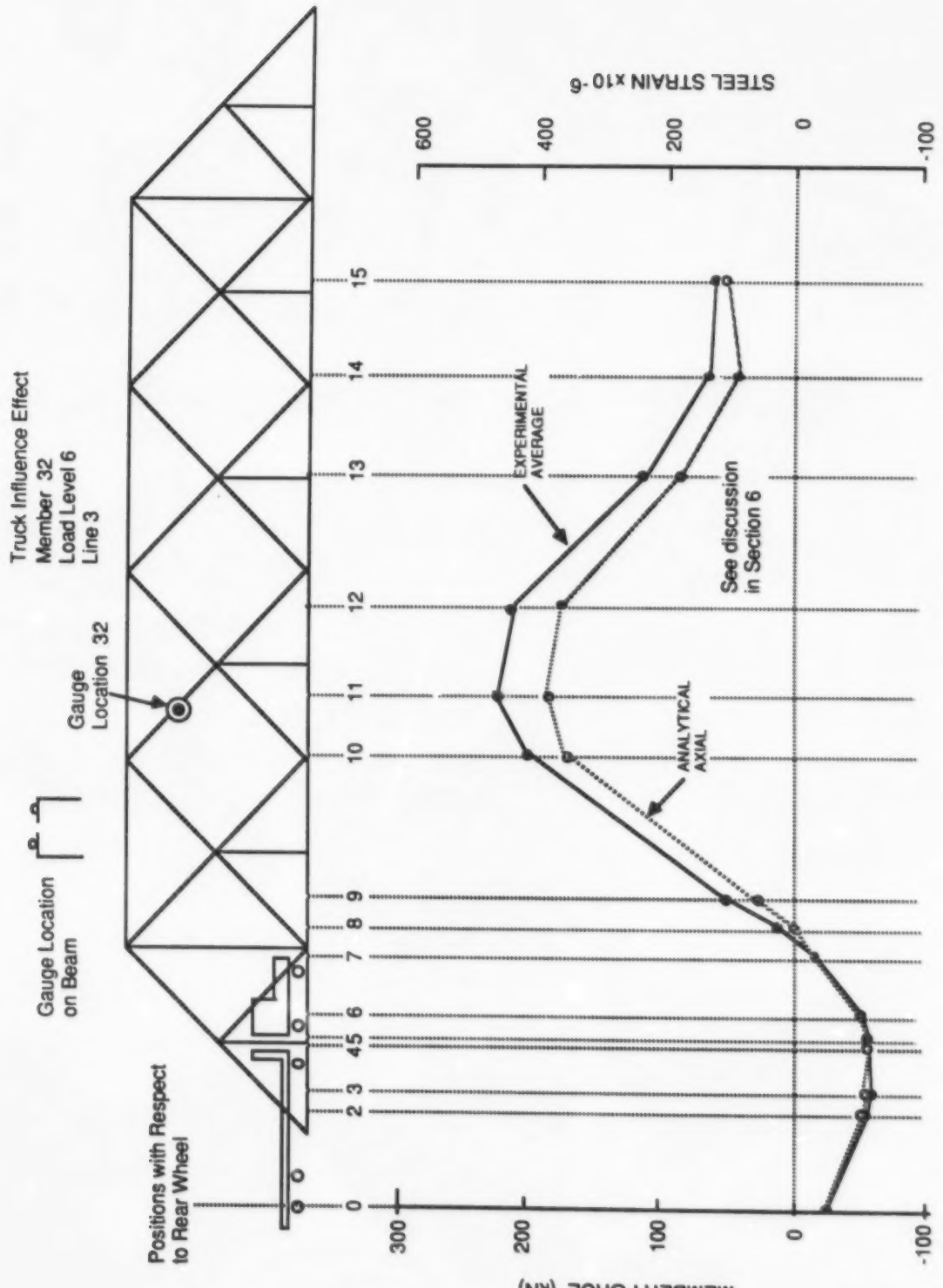


Figure 48/ Member Forces and Strains–Steel Truss, Span No. 1, Member 32 T_i, Load Level 6

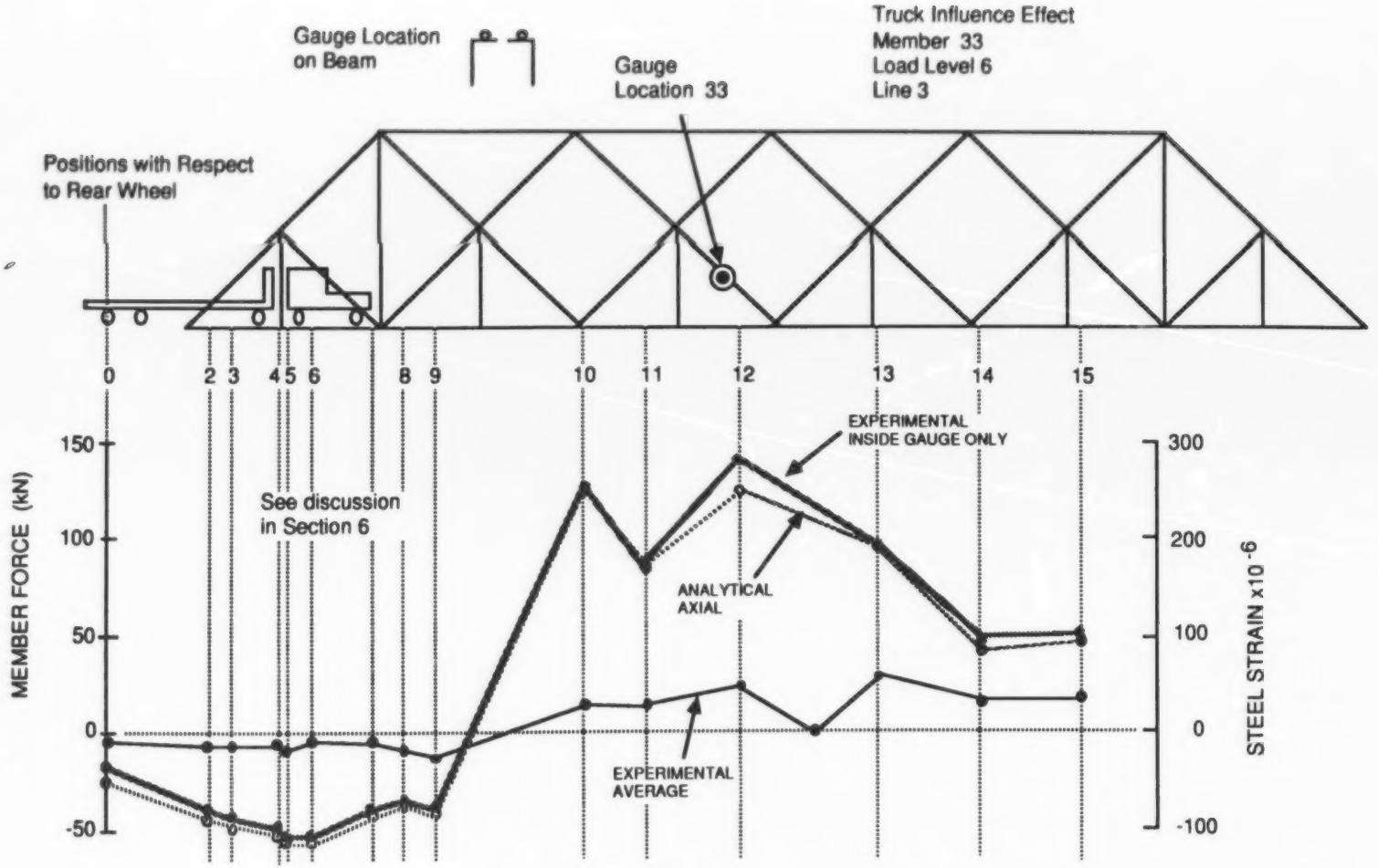


Figure 49/ Member Forces and Strains–Steel Truss, Span No. 1, Member 33, Load Level 6

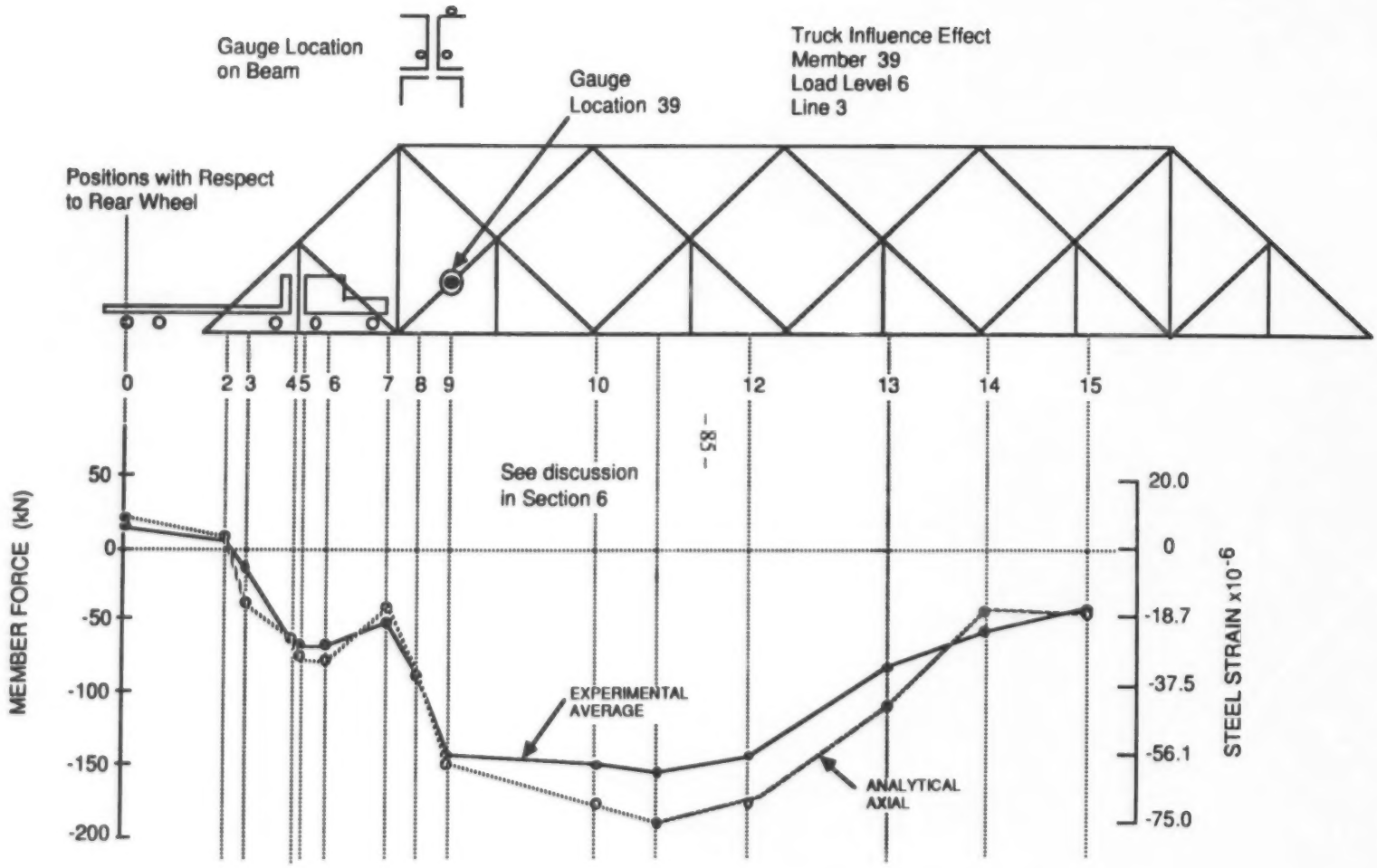


Figure 50/ Member Forces and Strains–Steel Truss, Span No. 1, Member 39, Load Level 6

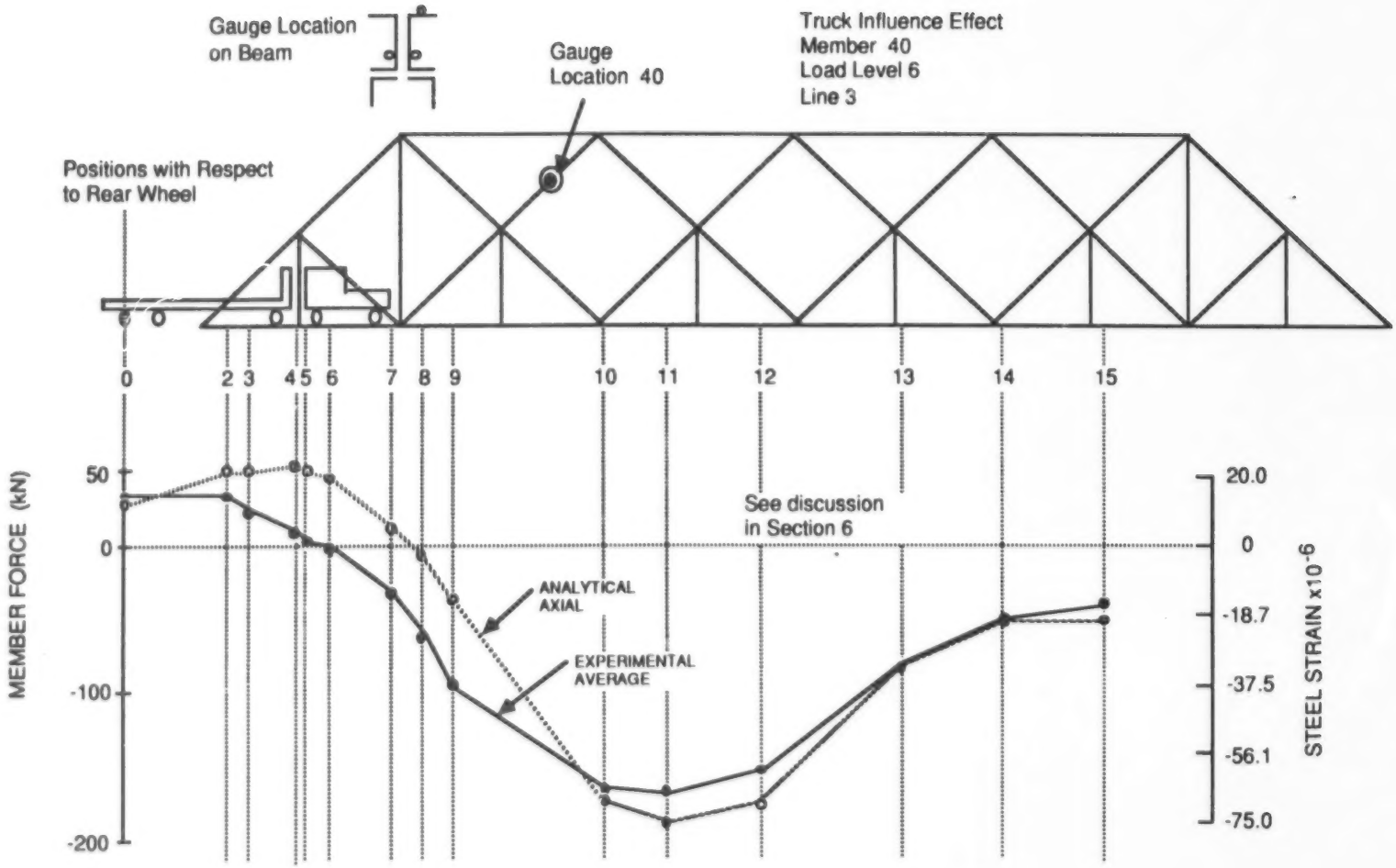


Figure 51/ Member Forces and Strains–Steel Truss, Span No. 1, Member 40, Load Level 6

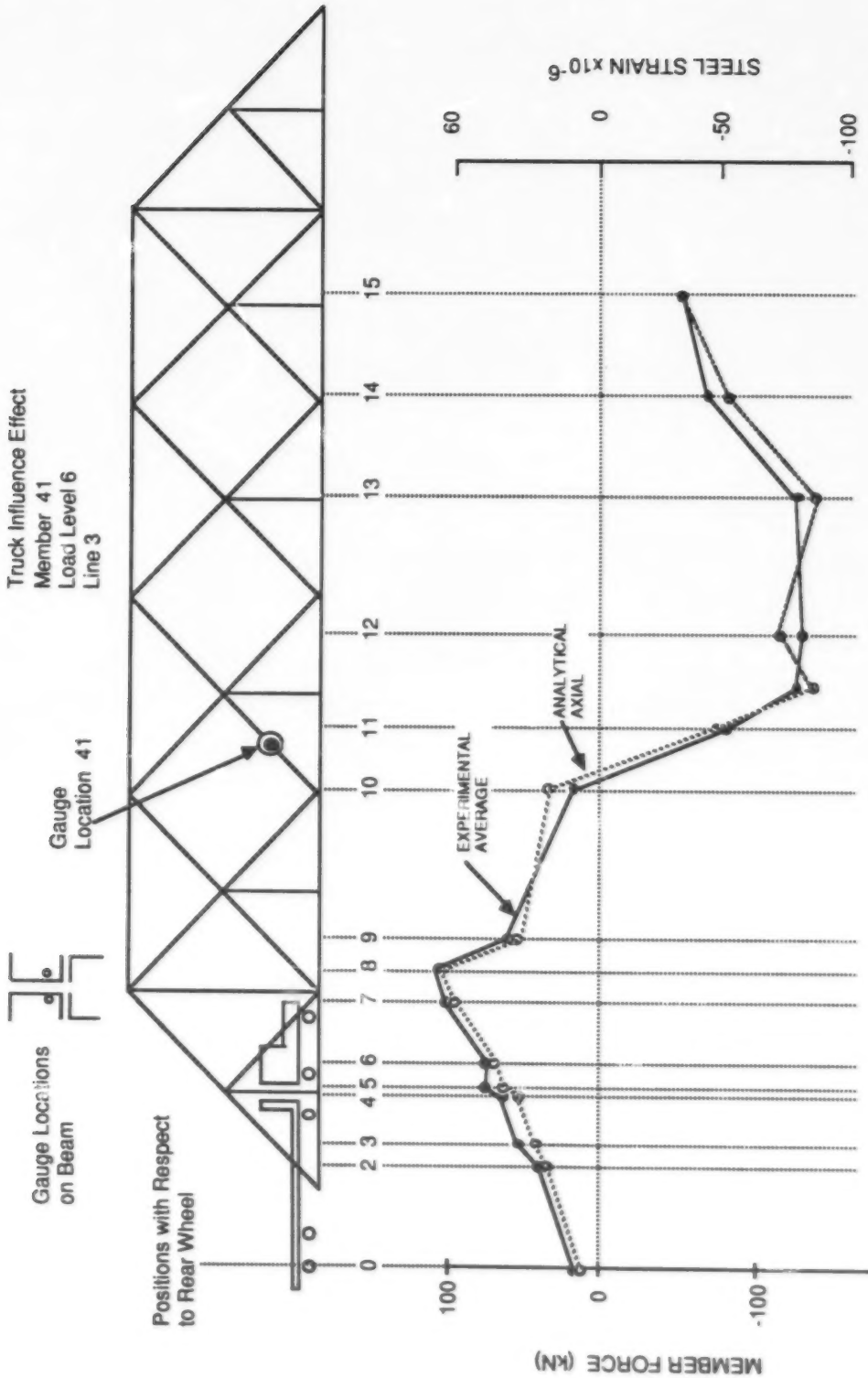


Figure 52/ Member Forces and Strains–Steel Truss, Span No. 1, Member 41, Load Level 6

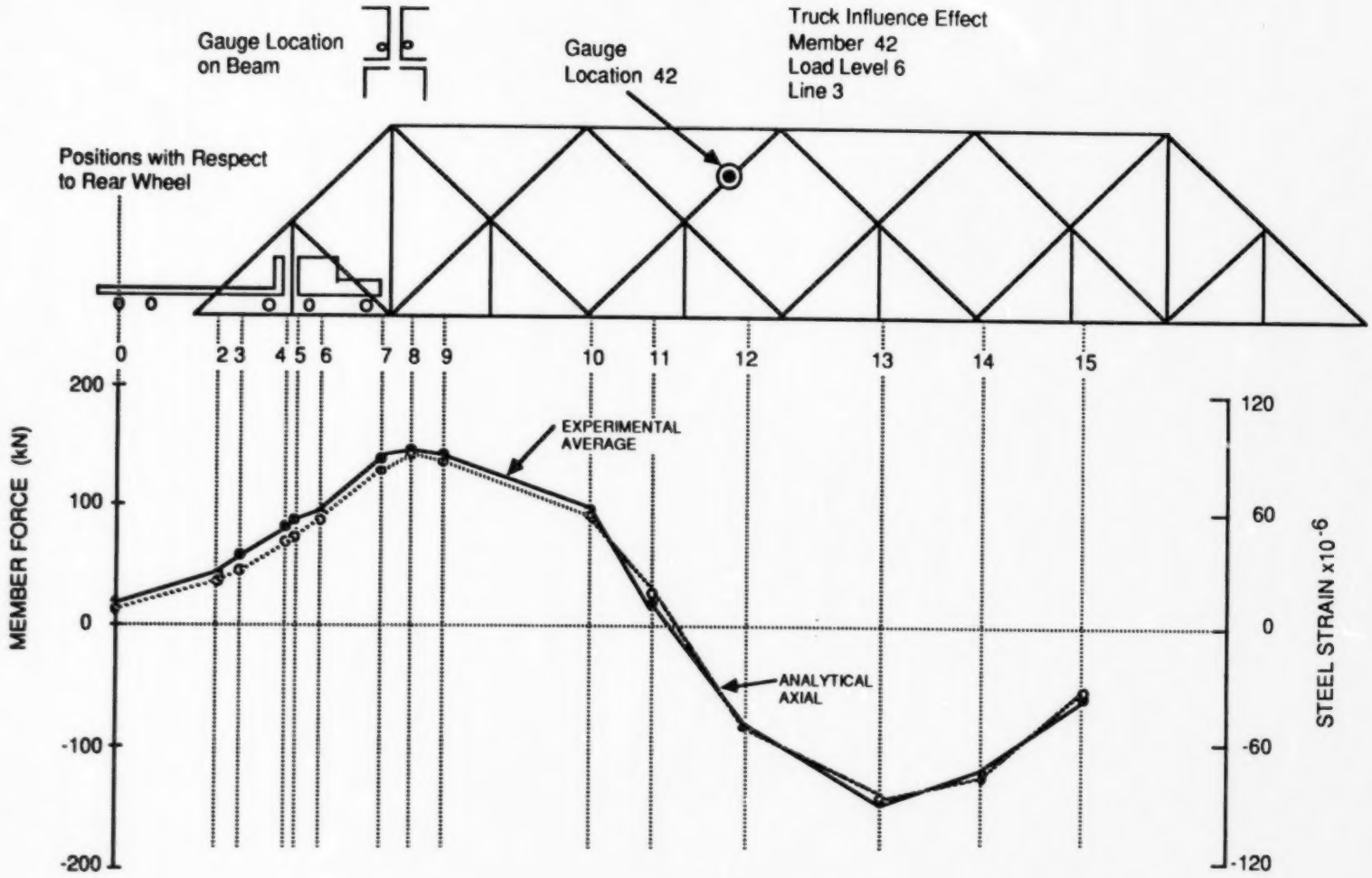


Figure 53/ Member Forces and Strains–Steel Truss, Span No. 1, Member 42, Load Level 6

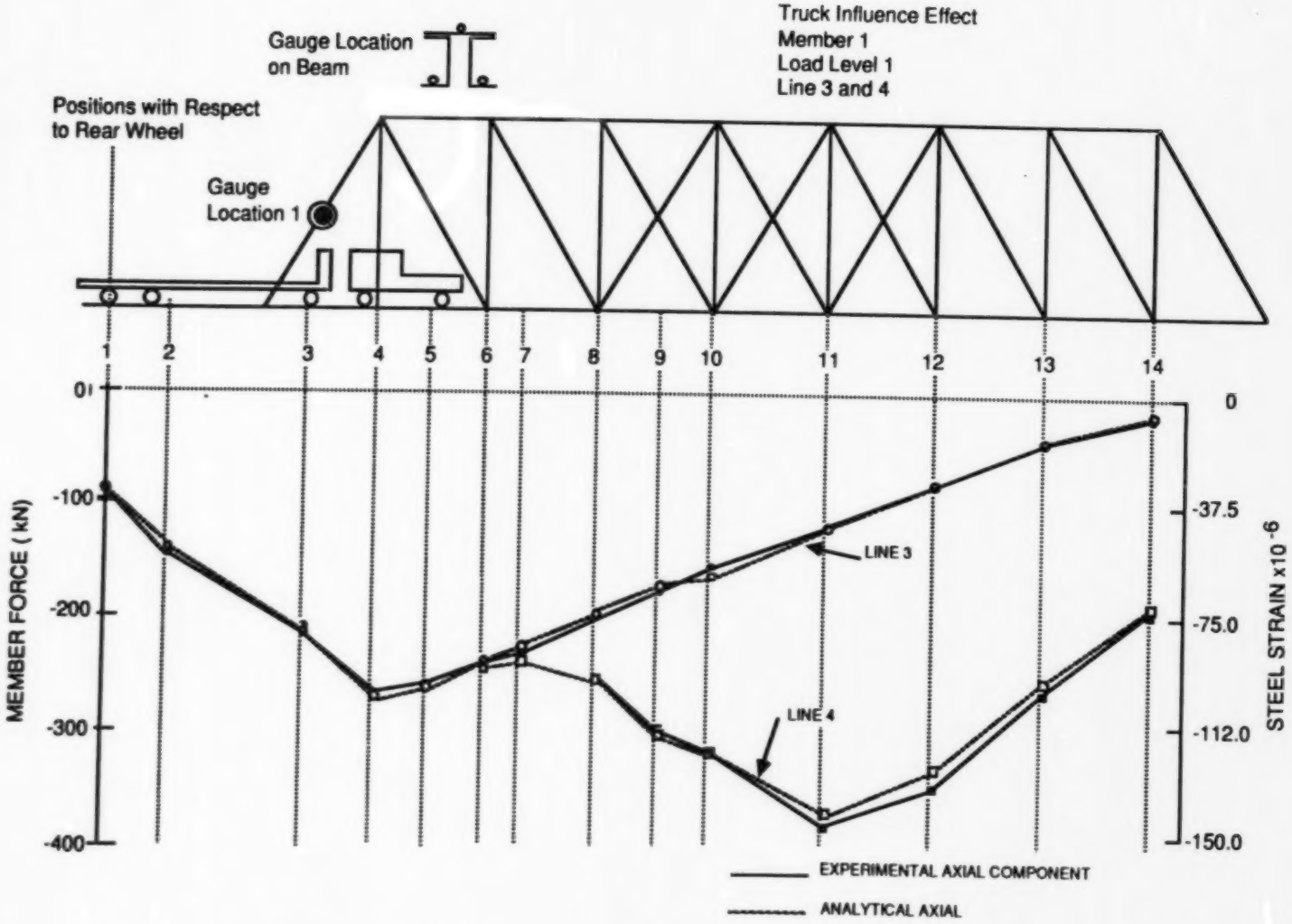


Figure 54/ Member Forces and Strains–Steel Truss, Span No. 2, Member 1, Load Level 1

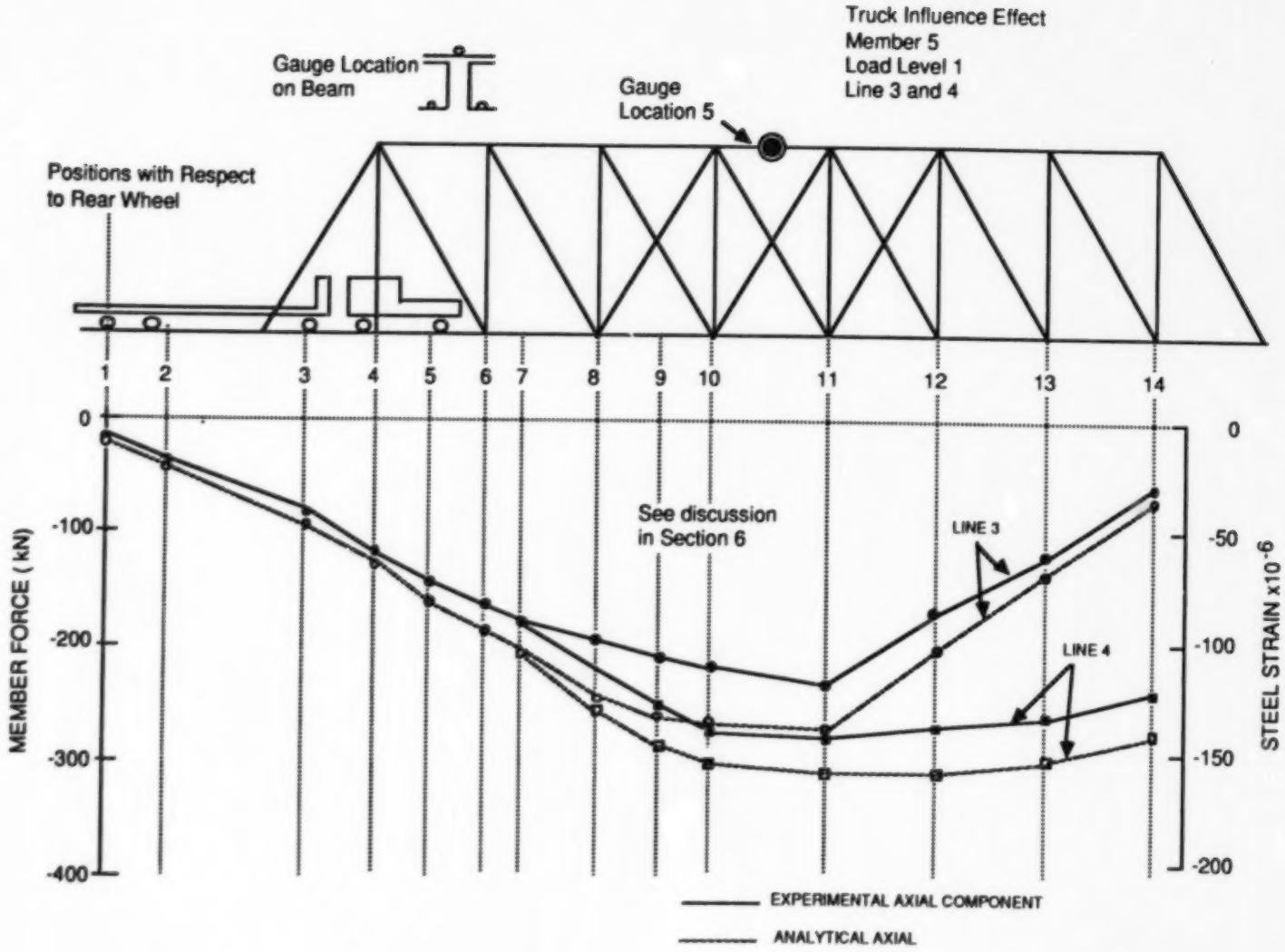


Figure 55/ Member Forces and Strains–Steel Truss, Span No. 2, Member 5, Load Level 1

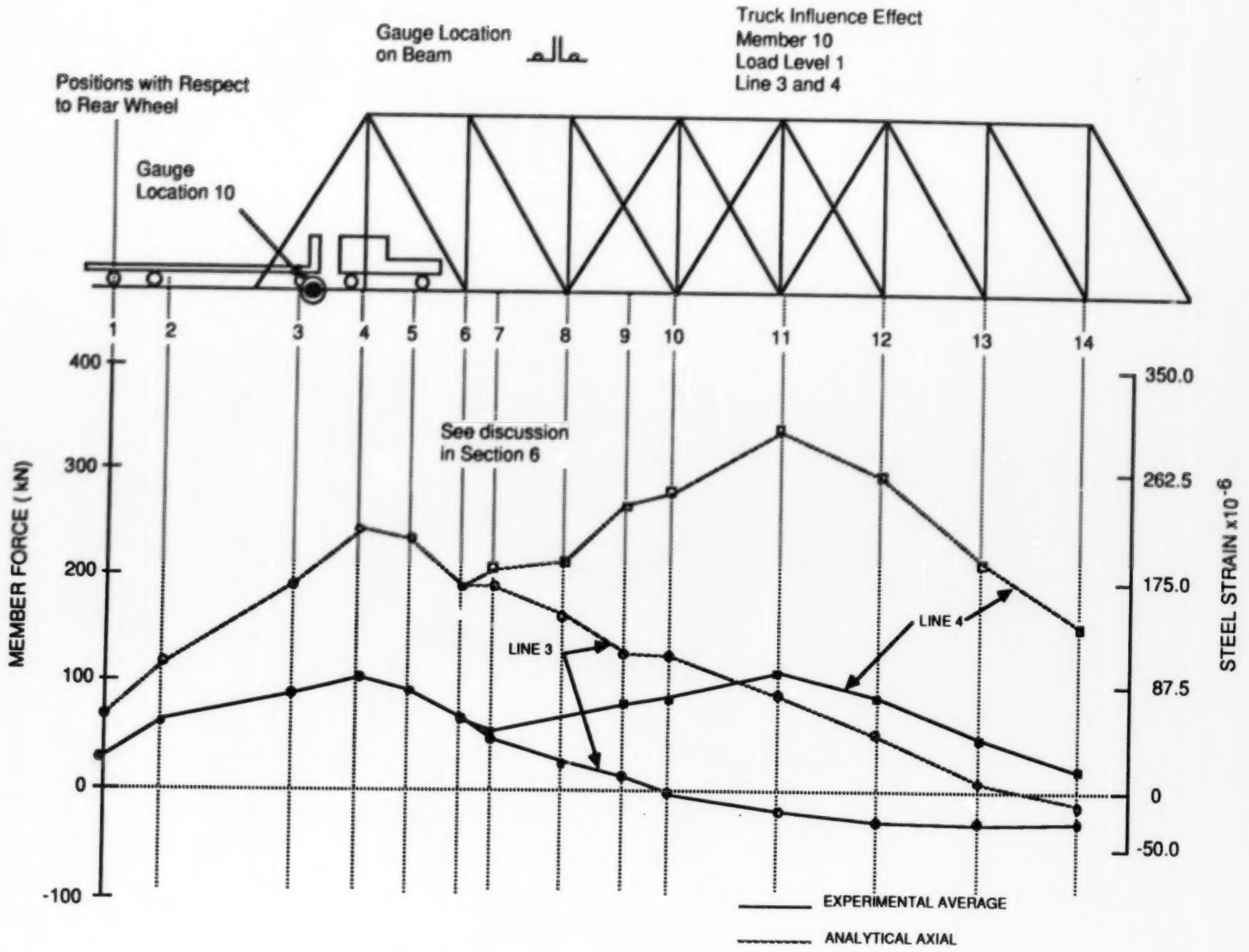


Figure 56/ Member Forces and Strains–Steel Truss, Span No. 2, Member 10, Load Level 1

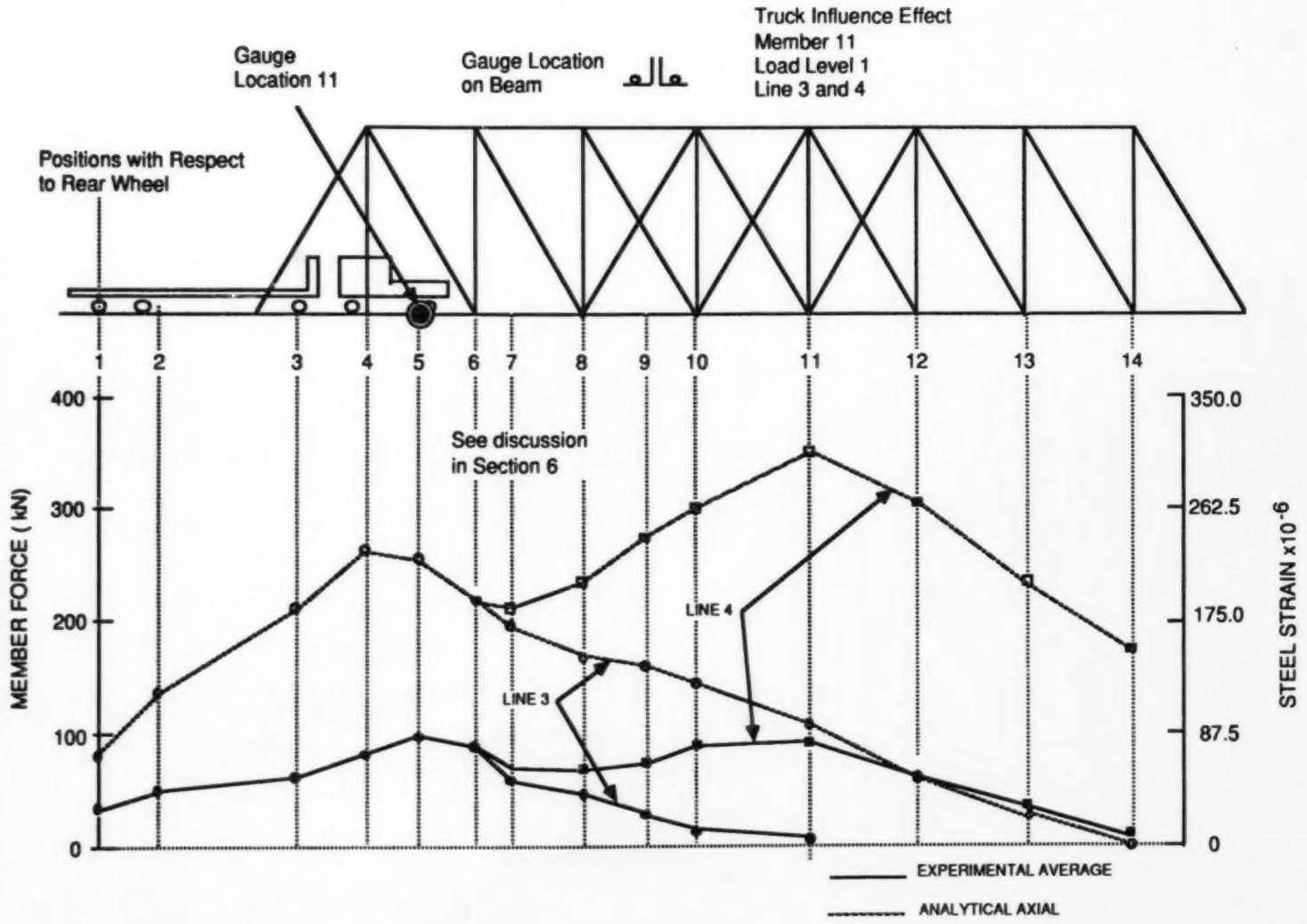


Figure 57/ Member Forces and Strains–Steel Truss, Span No. 2, Member 11, Load Level 1

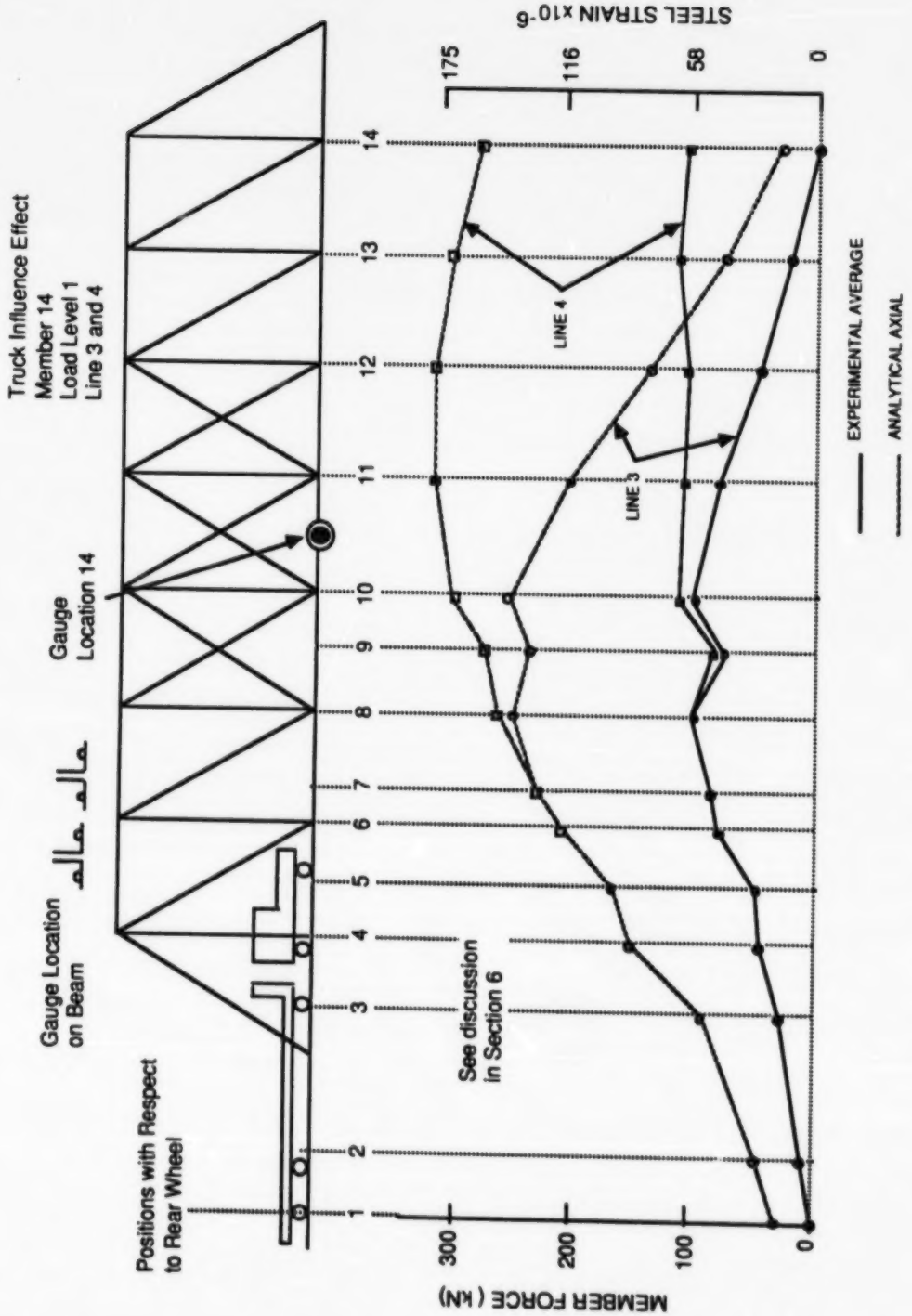


Figure 58/ Member Forces and Strains–Steel Truss, Span No. 2, Member 14, Load Level 1

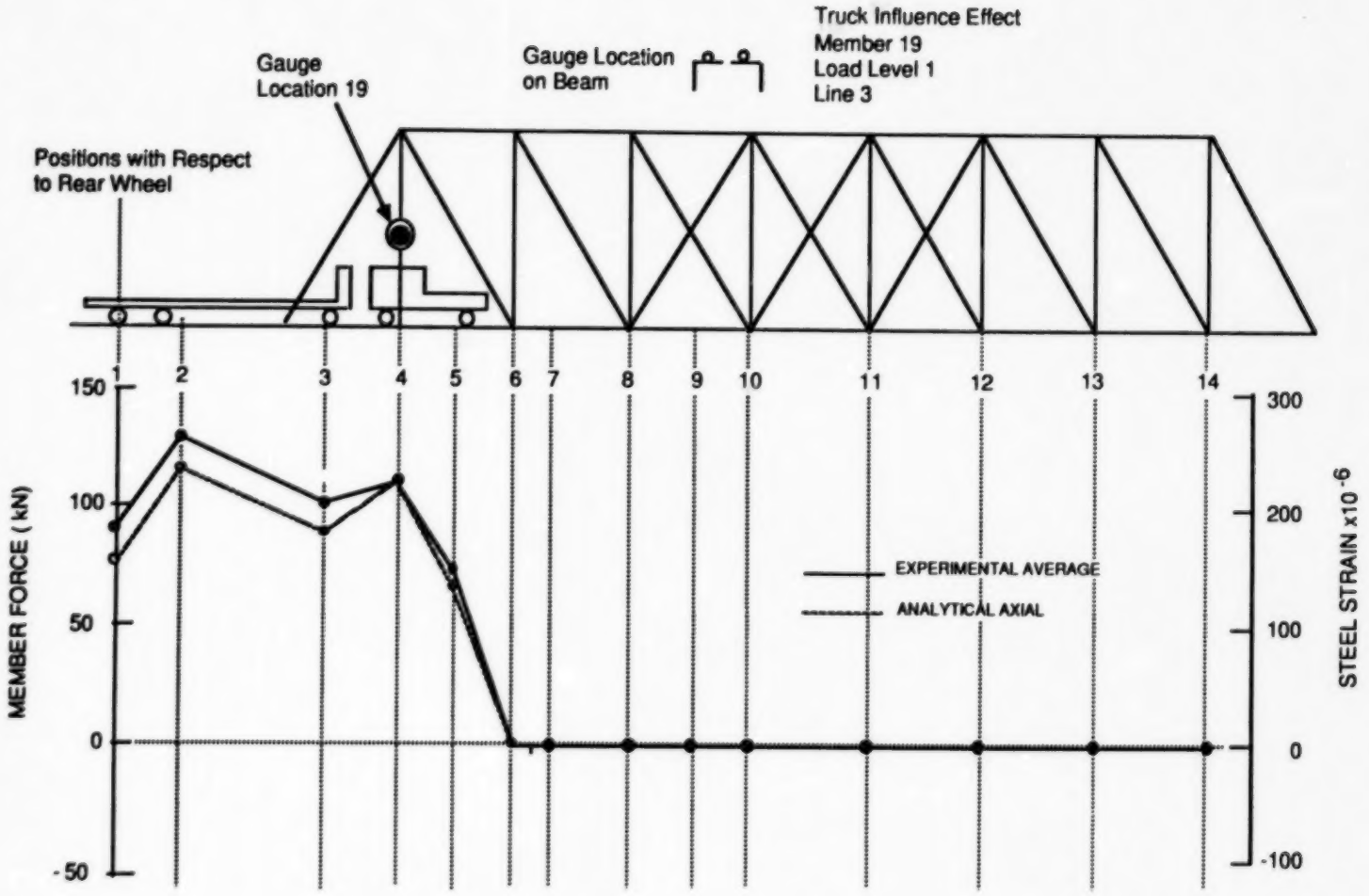


Figure 59/ Member Forces and Strains–Steel Truss, Span No. 2, Member 19, Load Level 1

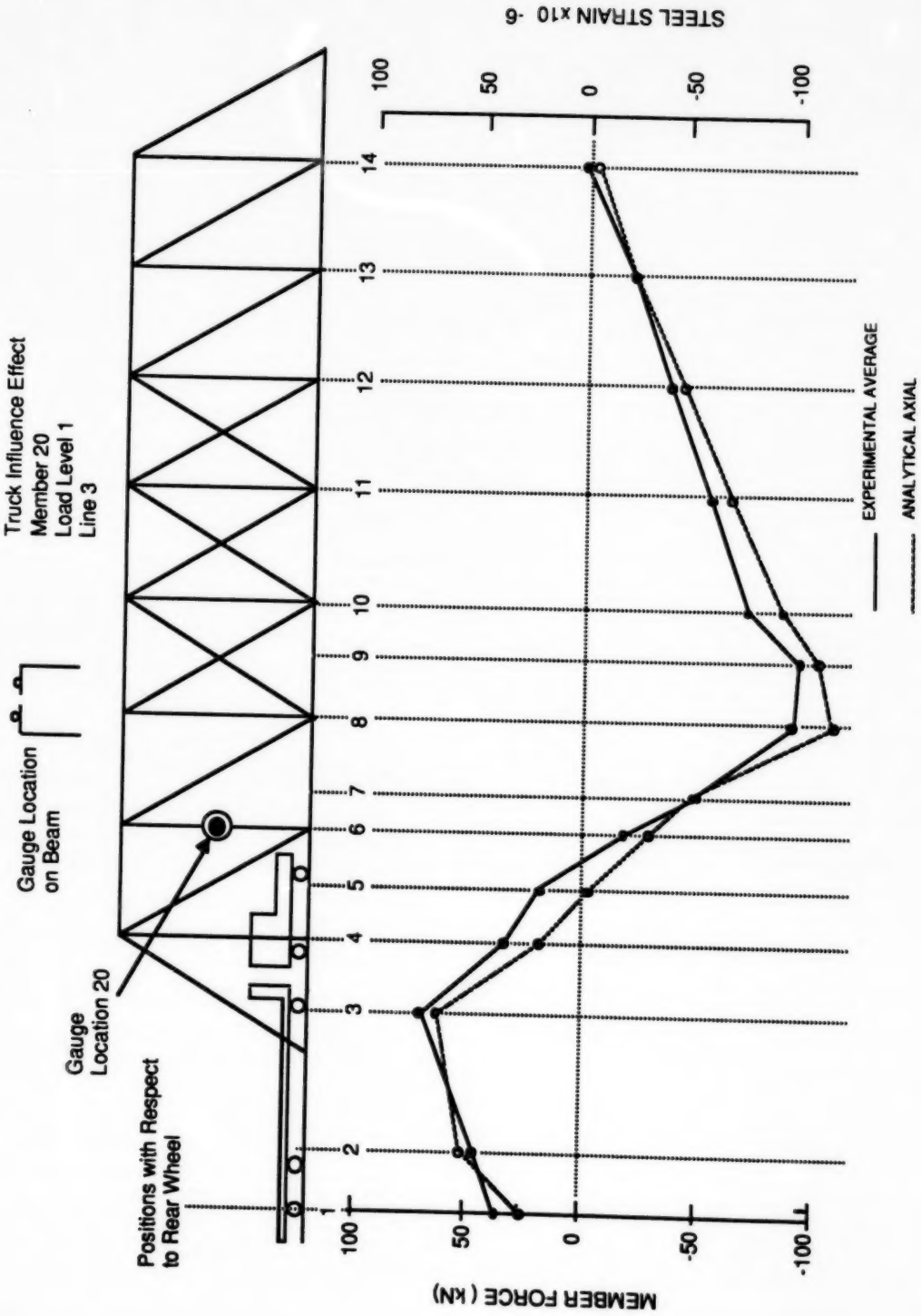


Figure 60/ Member Forces and Strains–Steel Truss, Span No. 2, Member 20, Load Level 1

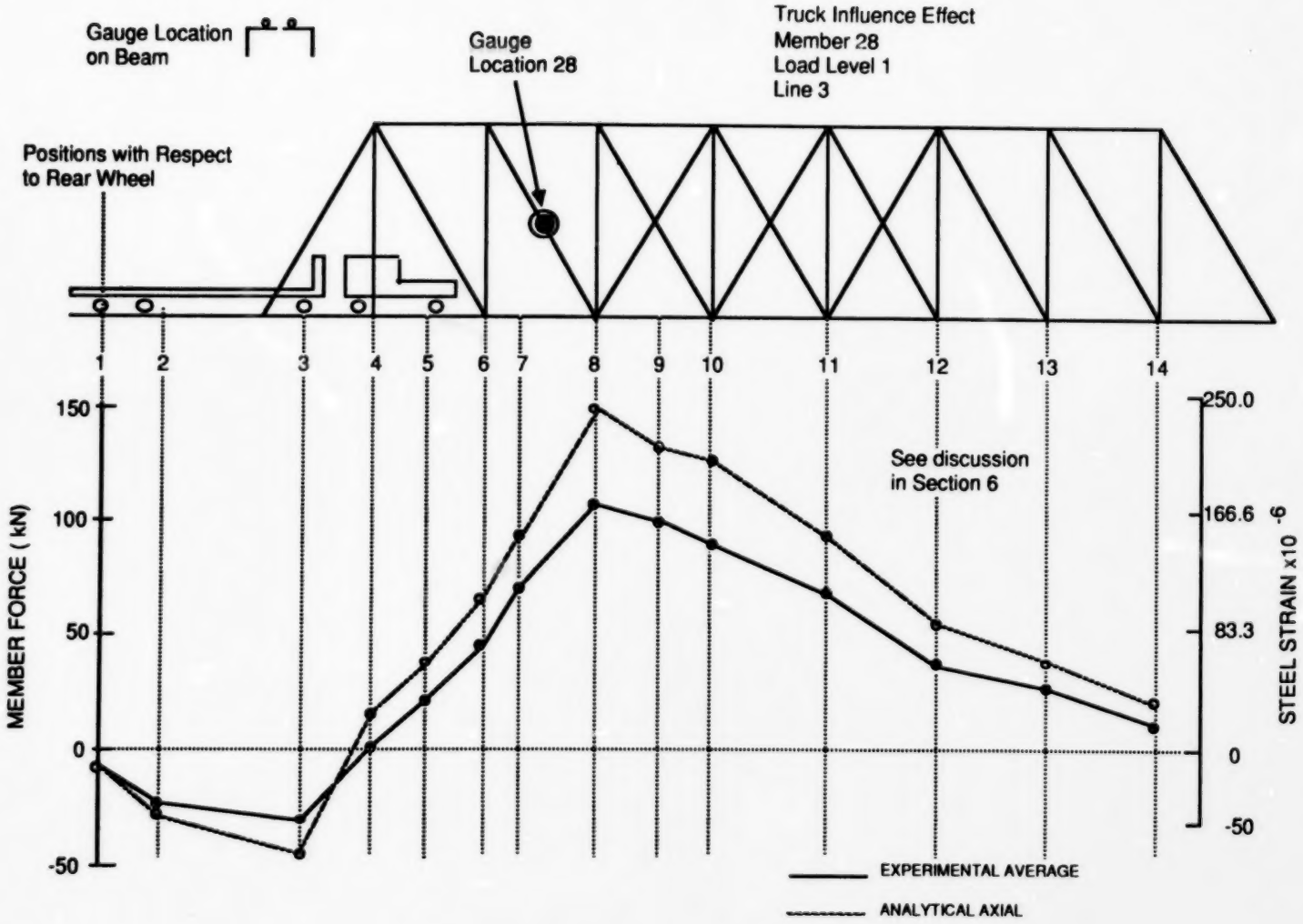


Figure 61/ Member Forces and Strains–Steel Truss, Span No. 2, Member 27, Load Level 1

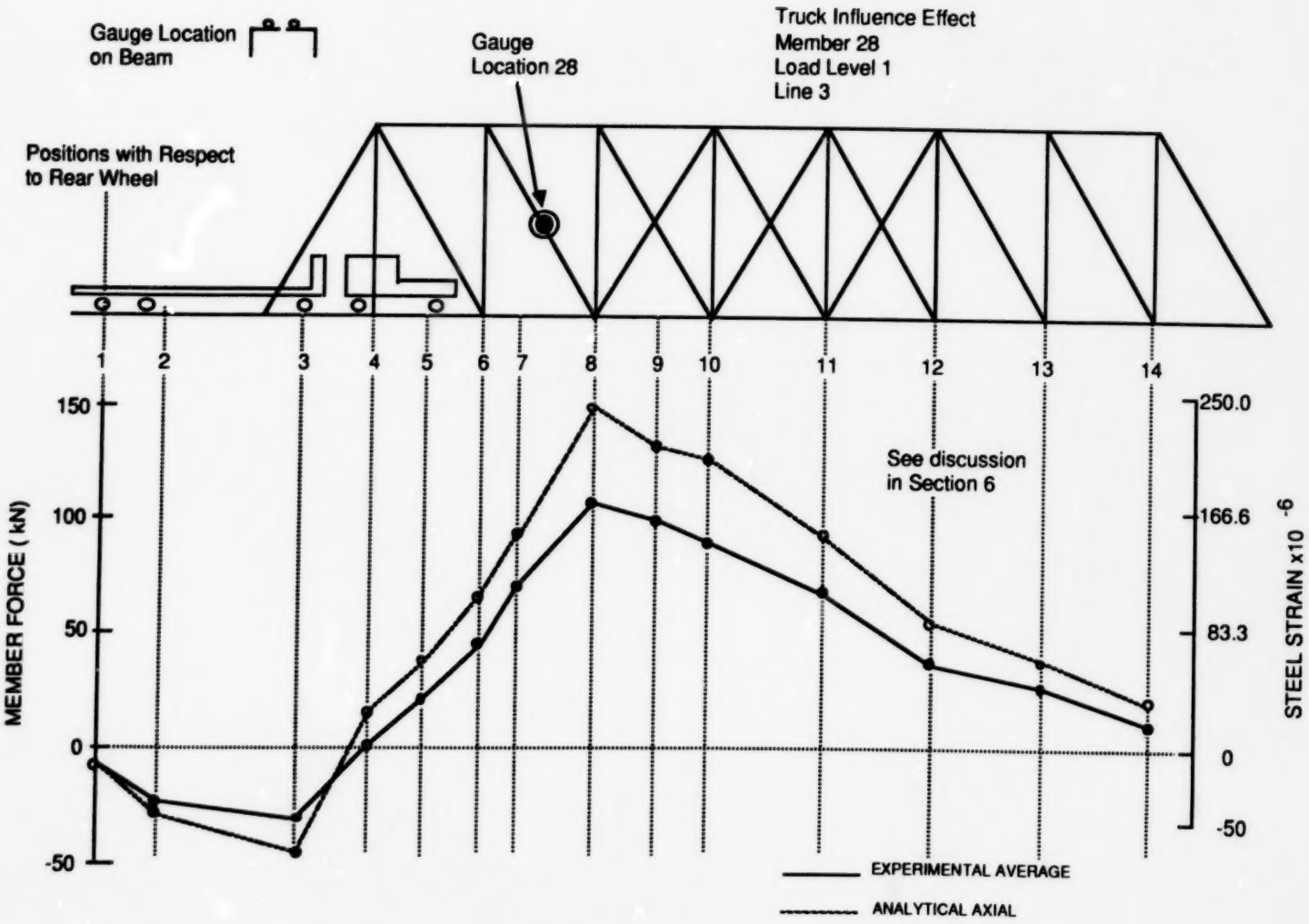


Figure 62/ Member Forces and Strains–Steel Truss, Span No. 2, Member 28, Load Level 1

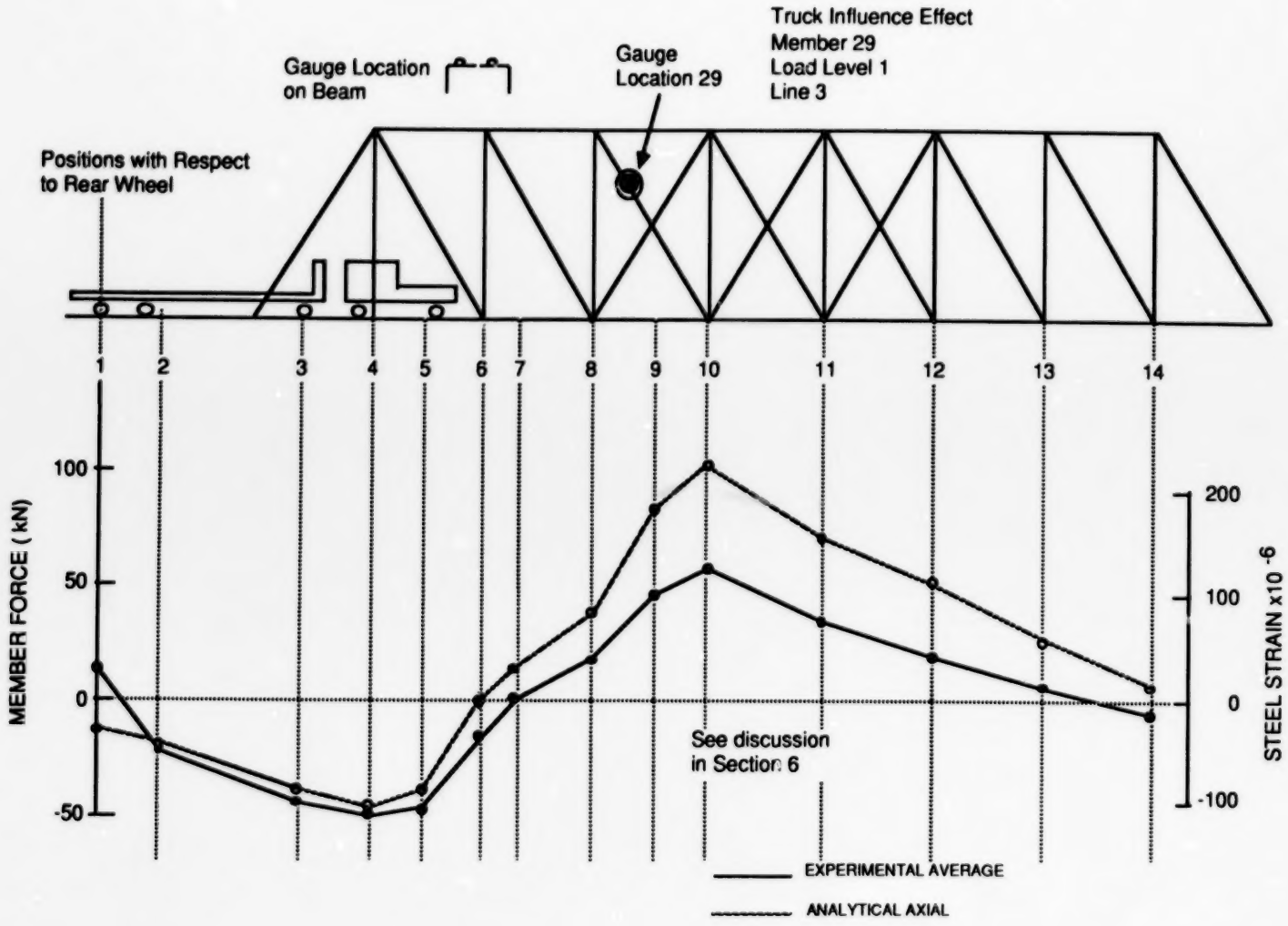


Figure 63/ Member Forces and Strains–Steel Truss, Span No. 2, Member 29, Load Level 1

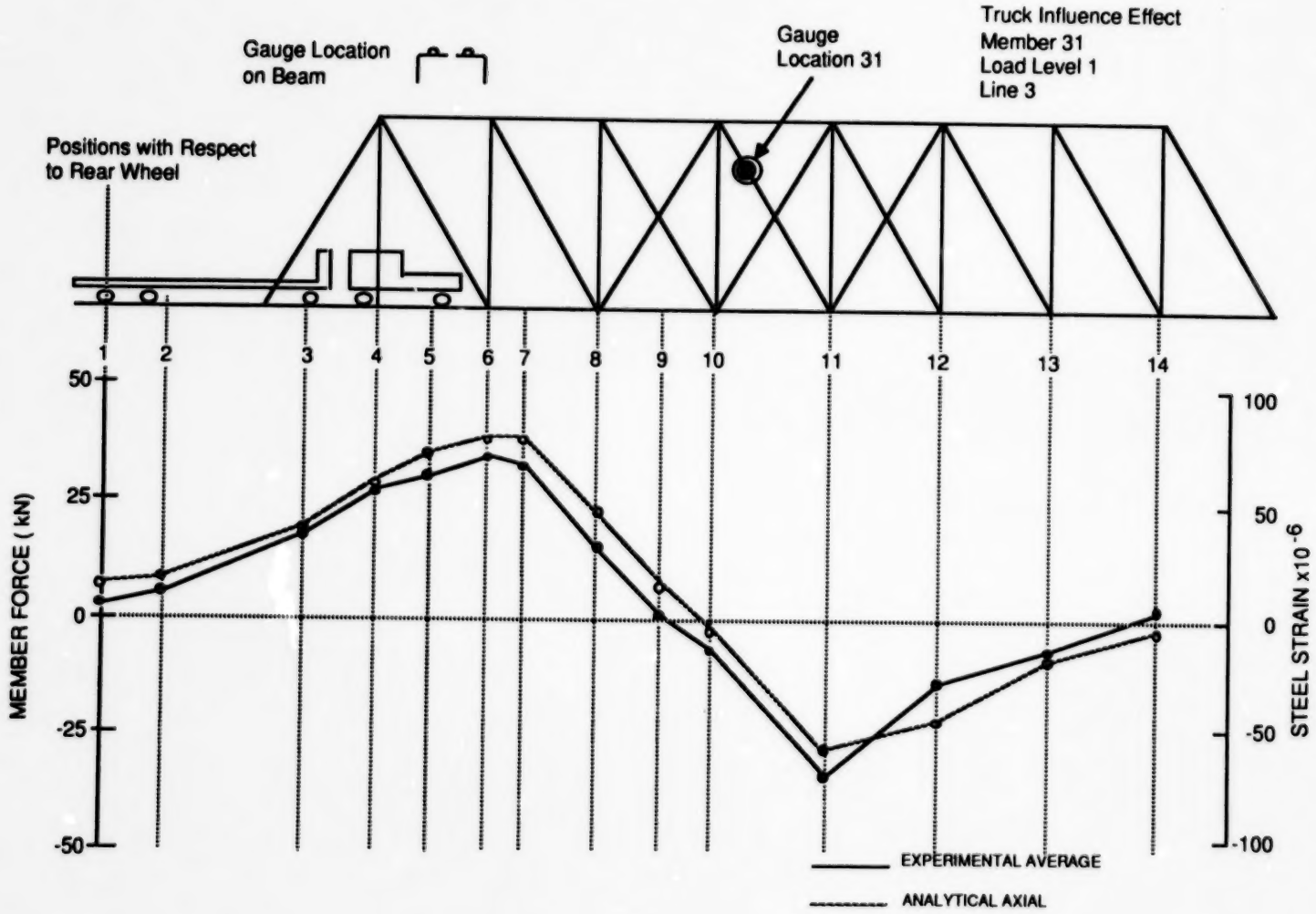


Figure 64/ Member Forces and Strains–Steel Truss, Span No. 2, Member 31, Load Level 1

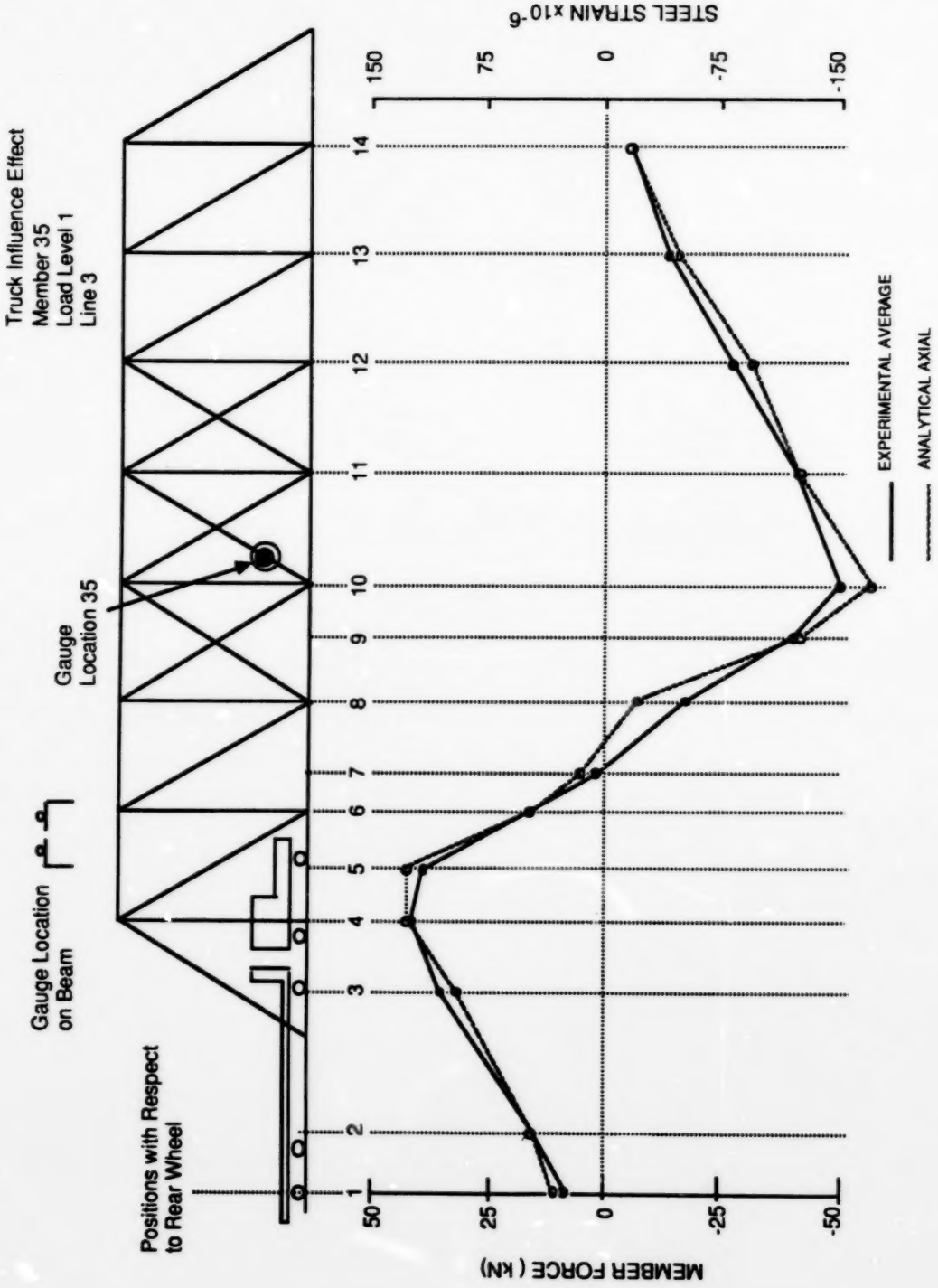


Figure 65/ Member Forces and Strains–Steel Truss, Span No. 2, Member 35, Load Level 1

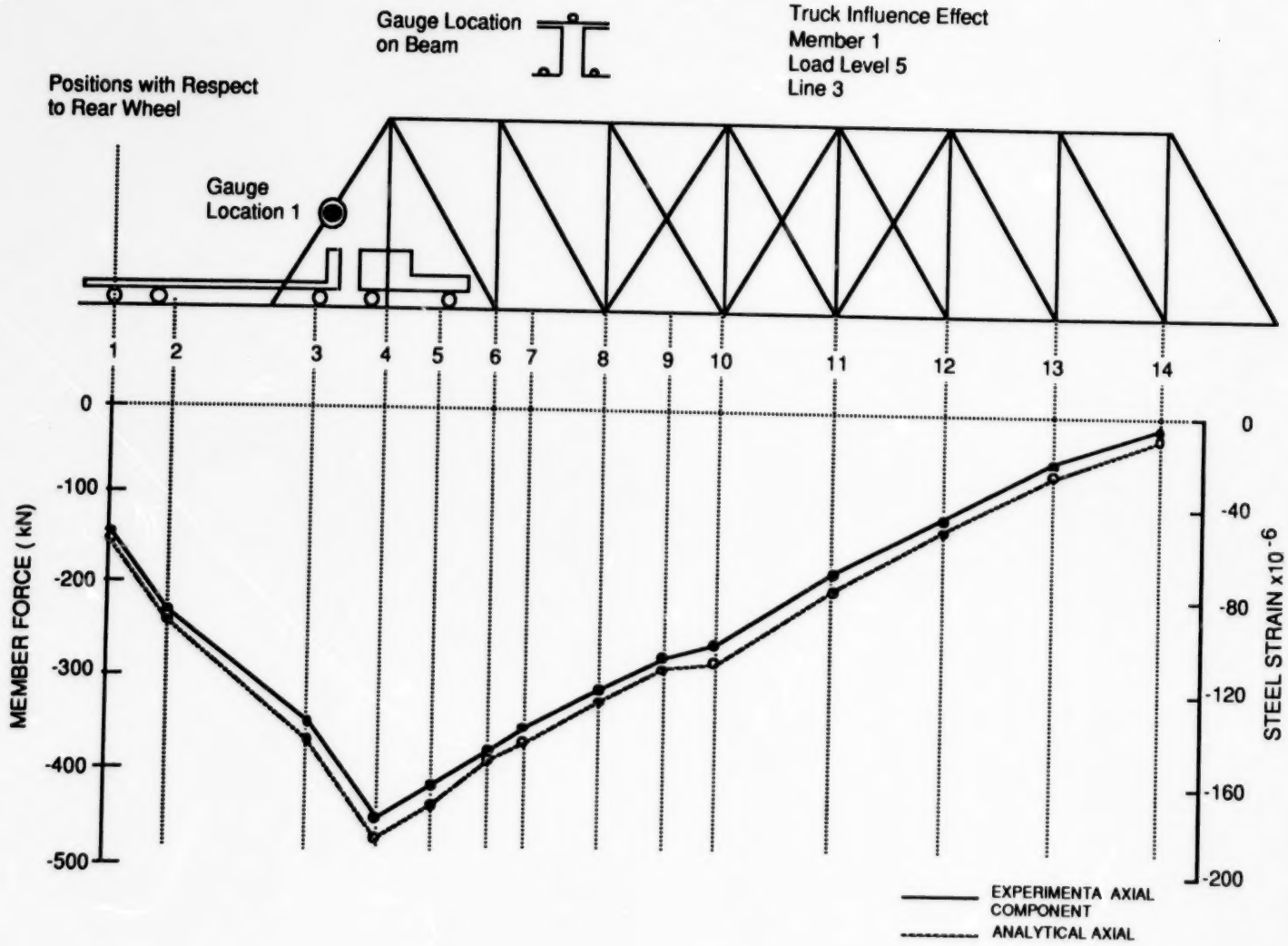


Figure 66/ Member Forces and Strains–Steel Truss, Span No. 2, Member 1, Load Level 5

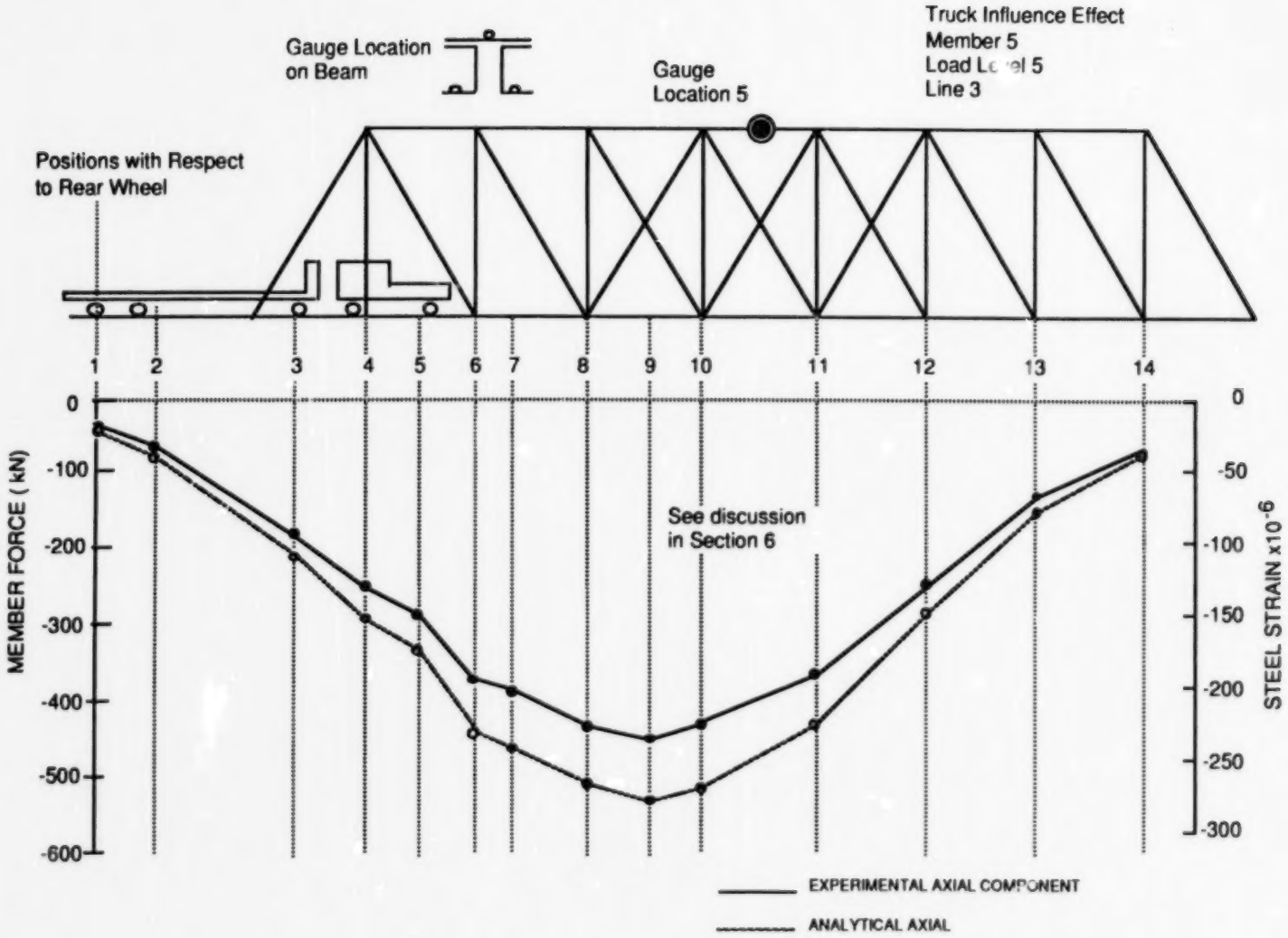


Figure 67/ Member Forces and Strains–Steel Truss, Span No. 2, Member 5, Load Level 5

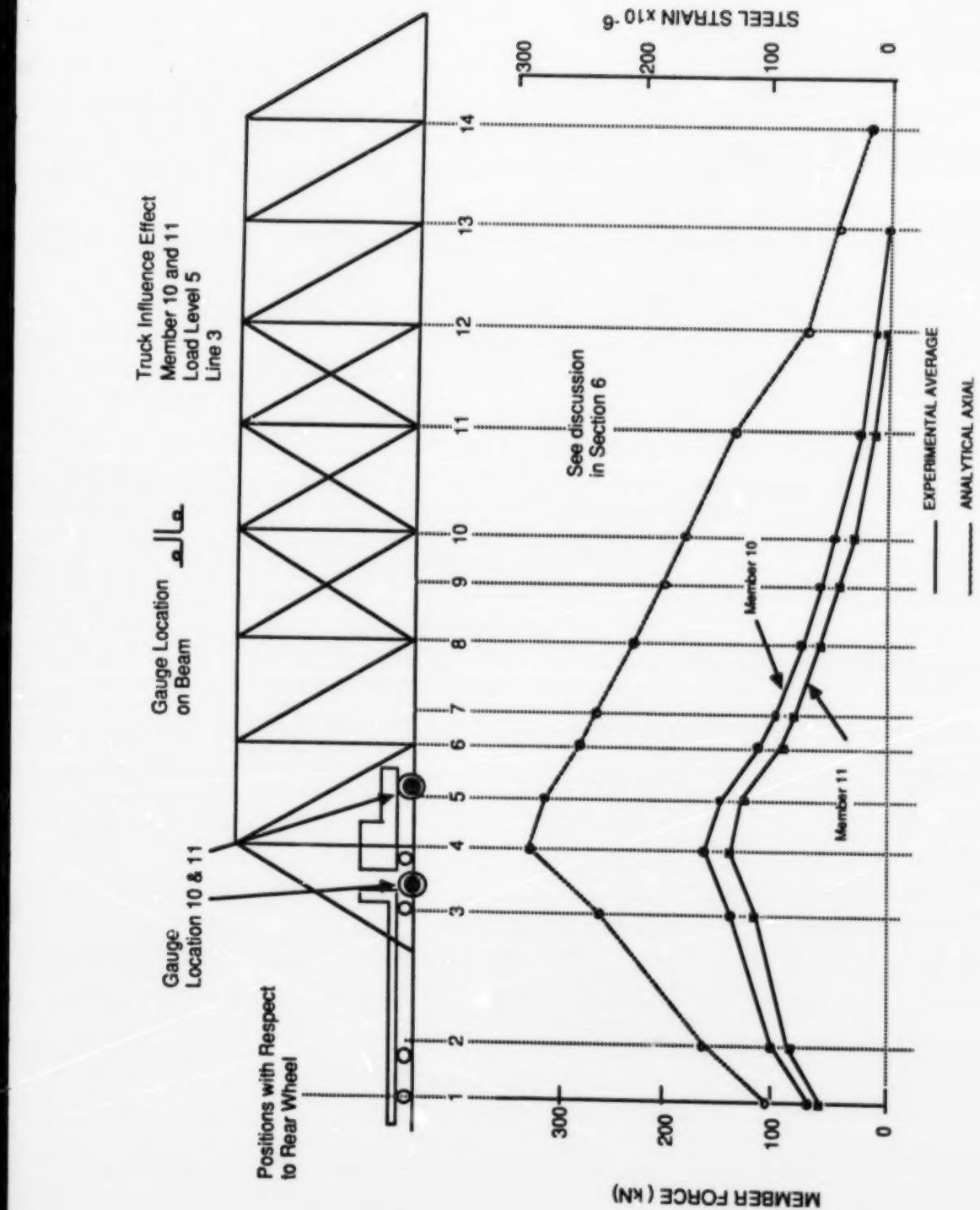


Figure 68/ Member Forces and Strains--Steel Truss, Span No. 2, Members 10 & 11, Load Level 5

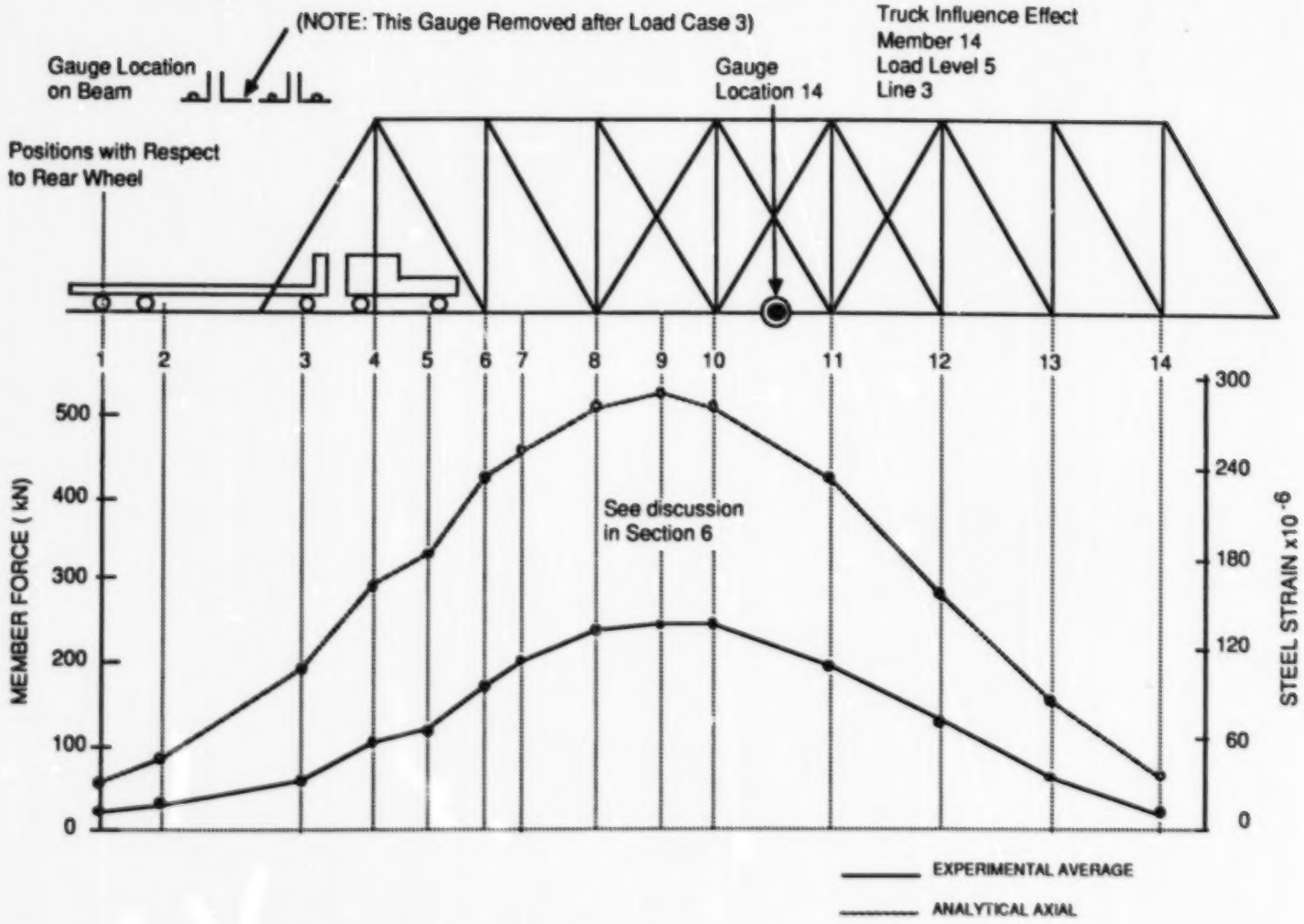


Figure 69/ Member Forces and Strains–Steel Truss, St No. 2, Member 14, Load Level 5

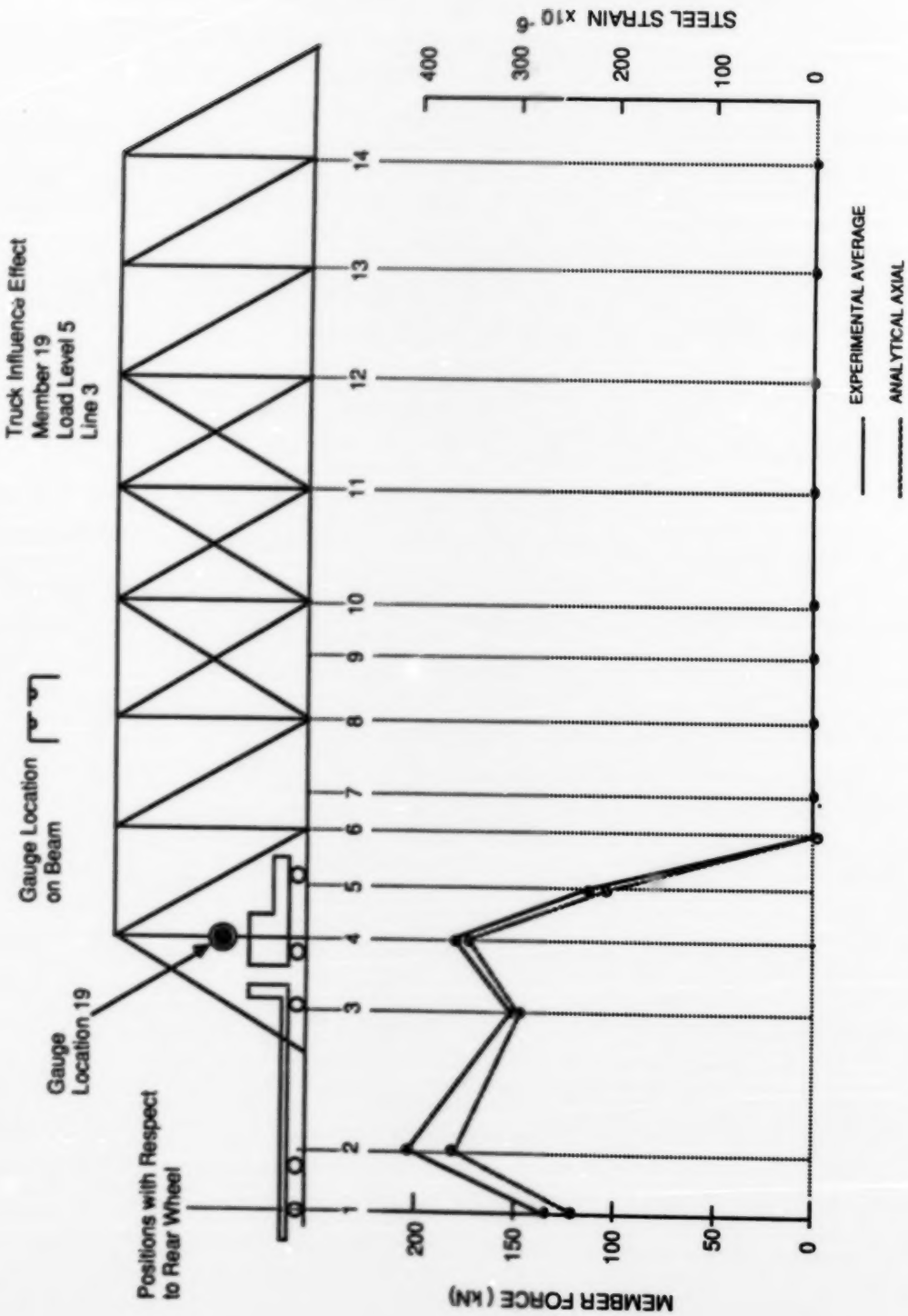


Figure 70/ Member Forces and Strains–Steel Truss, Span No. 2, Member 19, Load Level 5

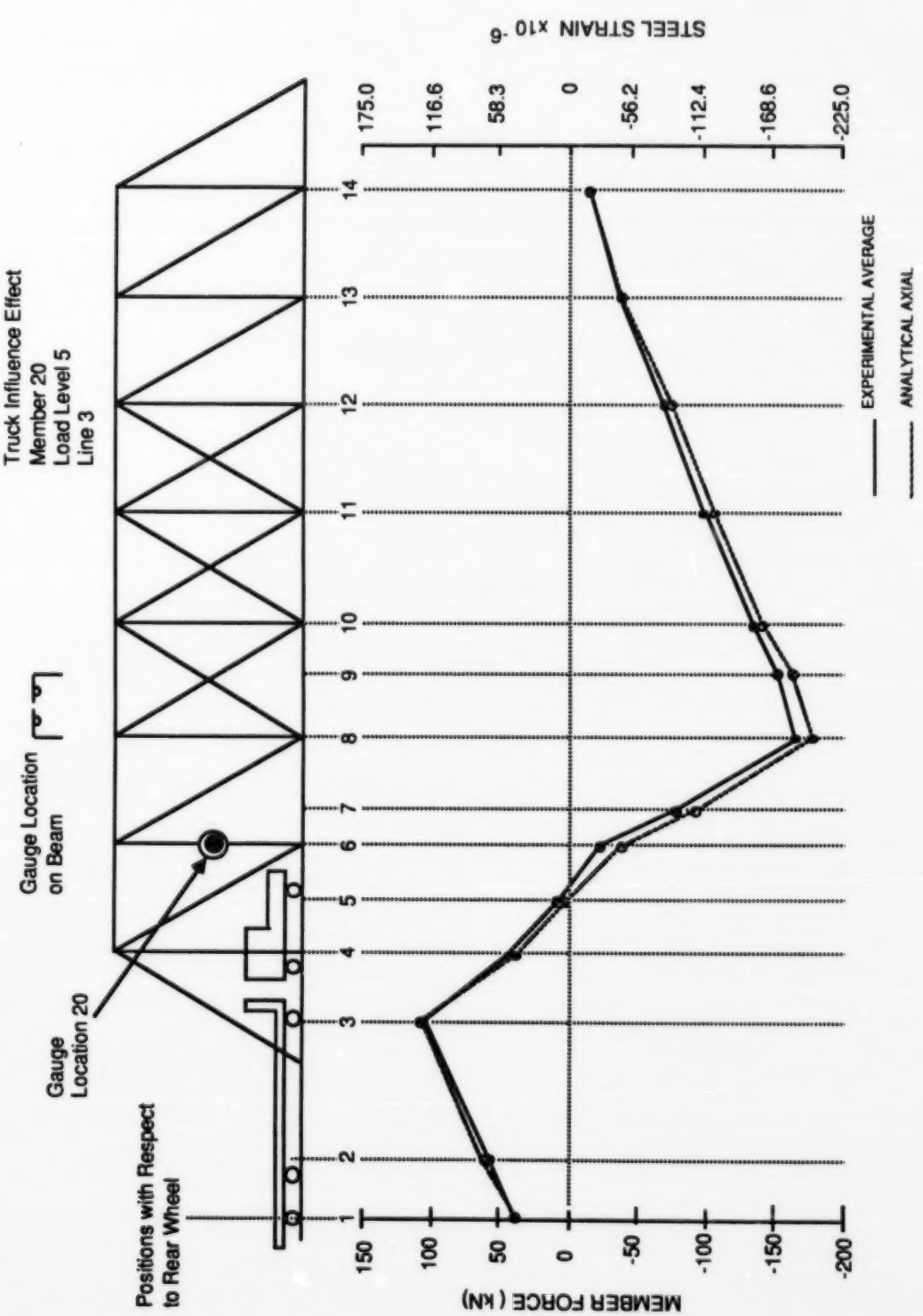


Figure 71/ Member Forces and Strains–Steel Truss, Span No. 2, Member 20, Load Level 5

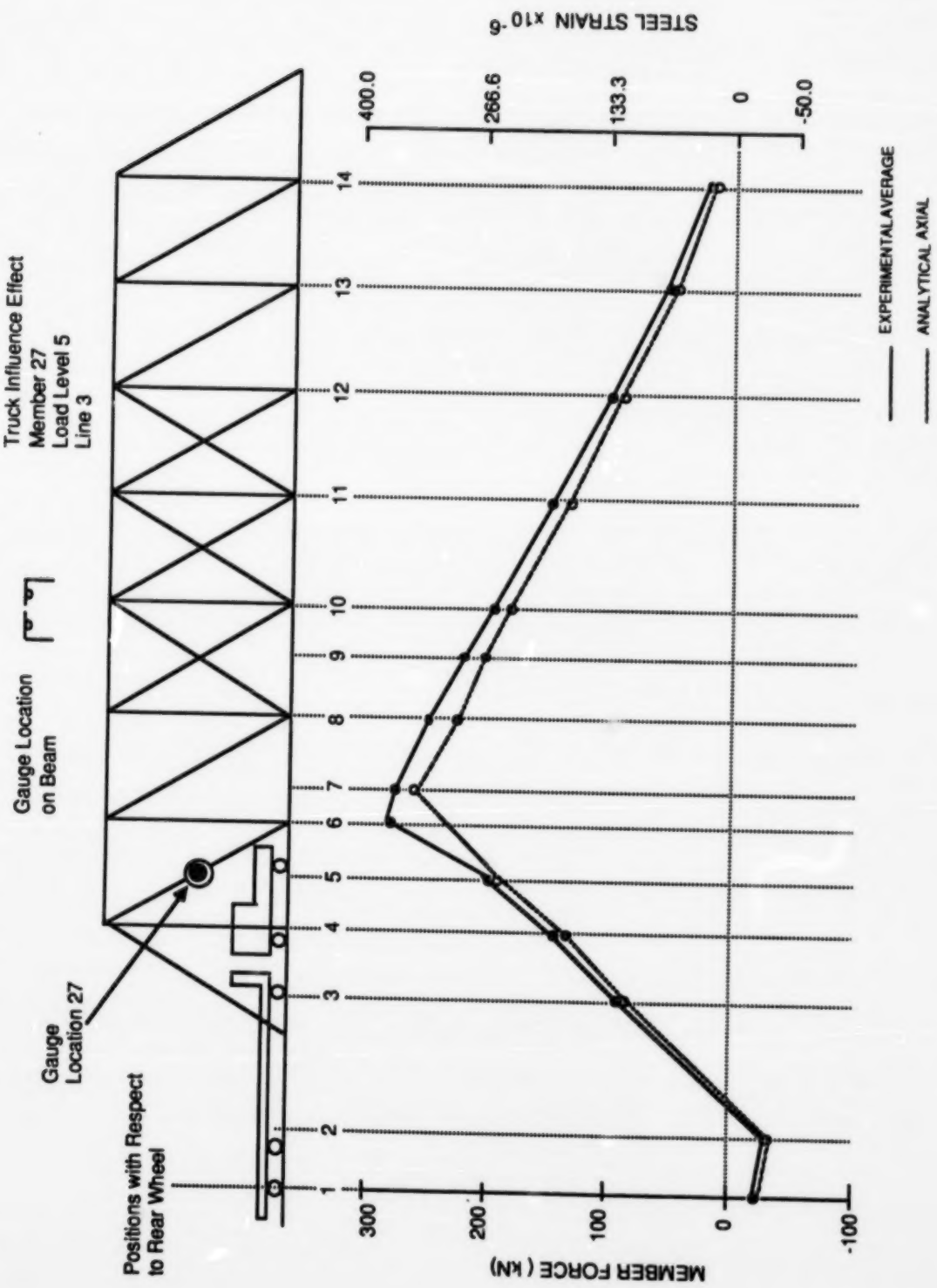


Figure 72/ Member Forces and Strains–Steel Truss, Span No. 2, Member 27, Load Level 5

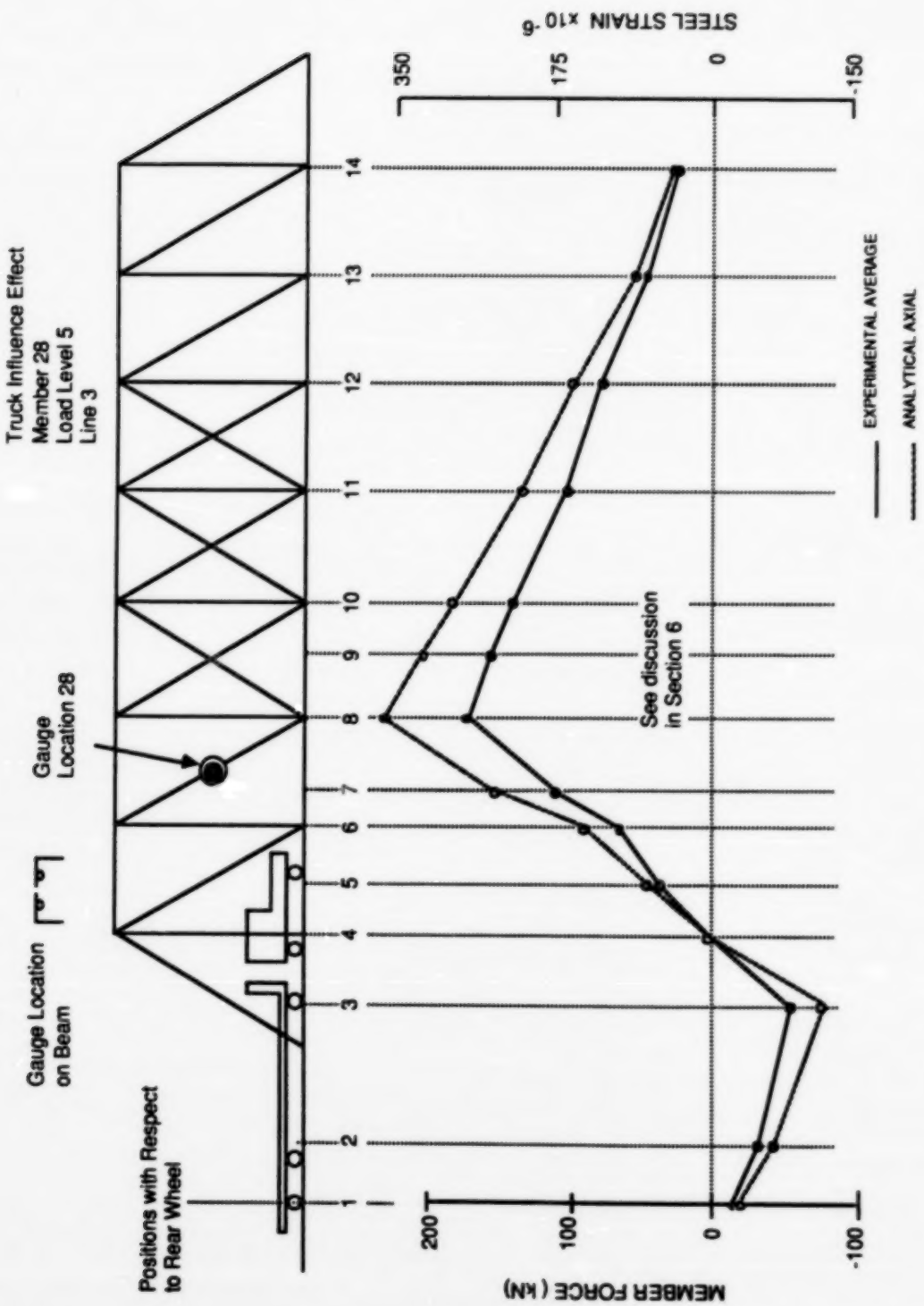


Figure 73/ Member Forces and Strains–Steel Truss, Span No. 2, Member 28, Load Level 5

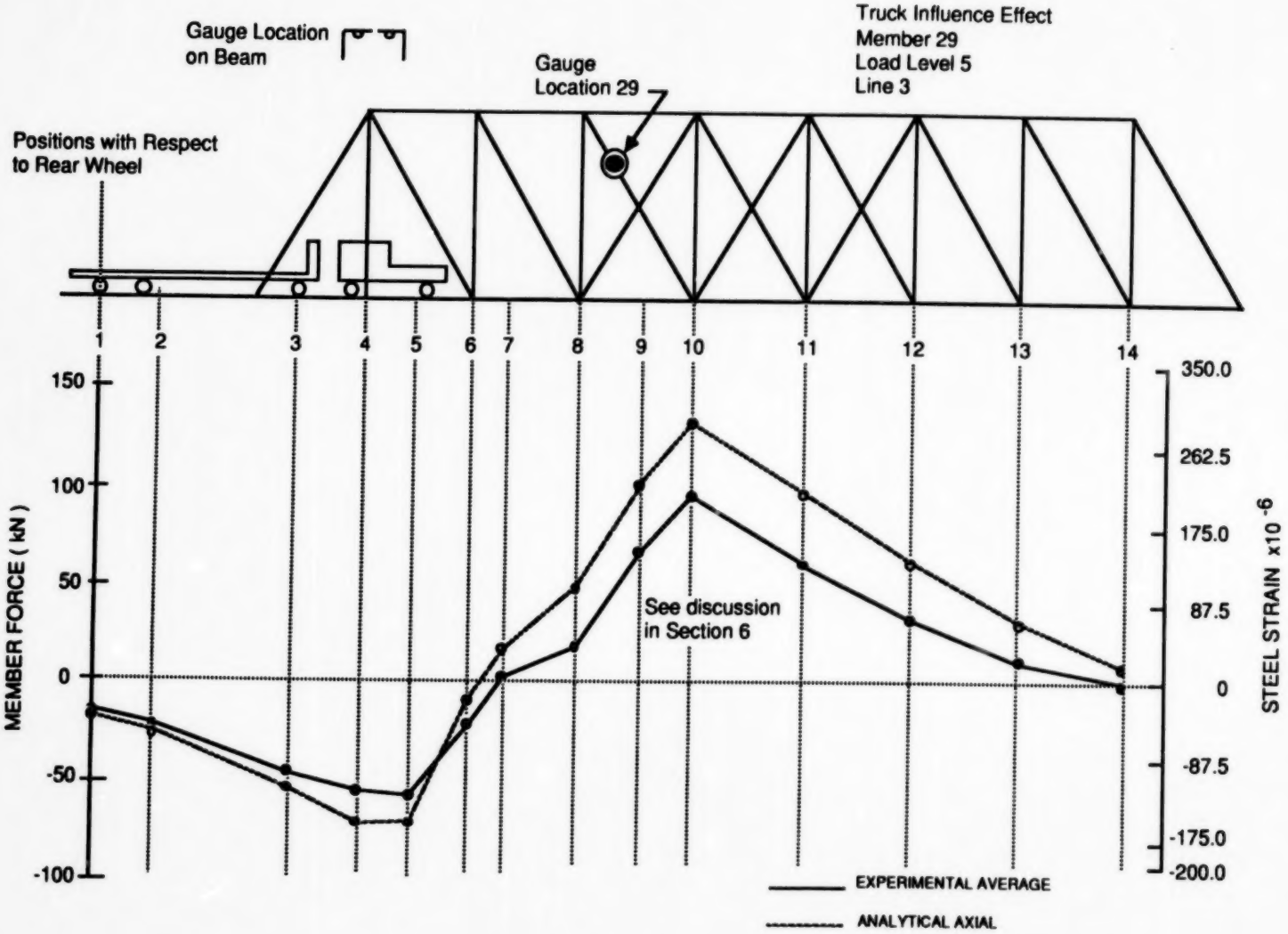


Figure 74/ Member Forces and Strains–Steel Truss, Span No. 2, Member 29, Load Level 5

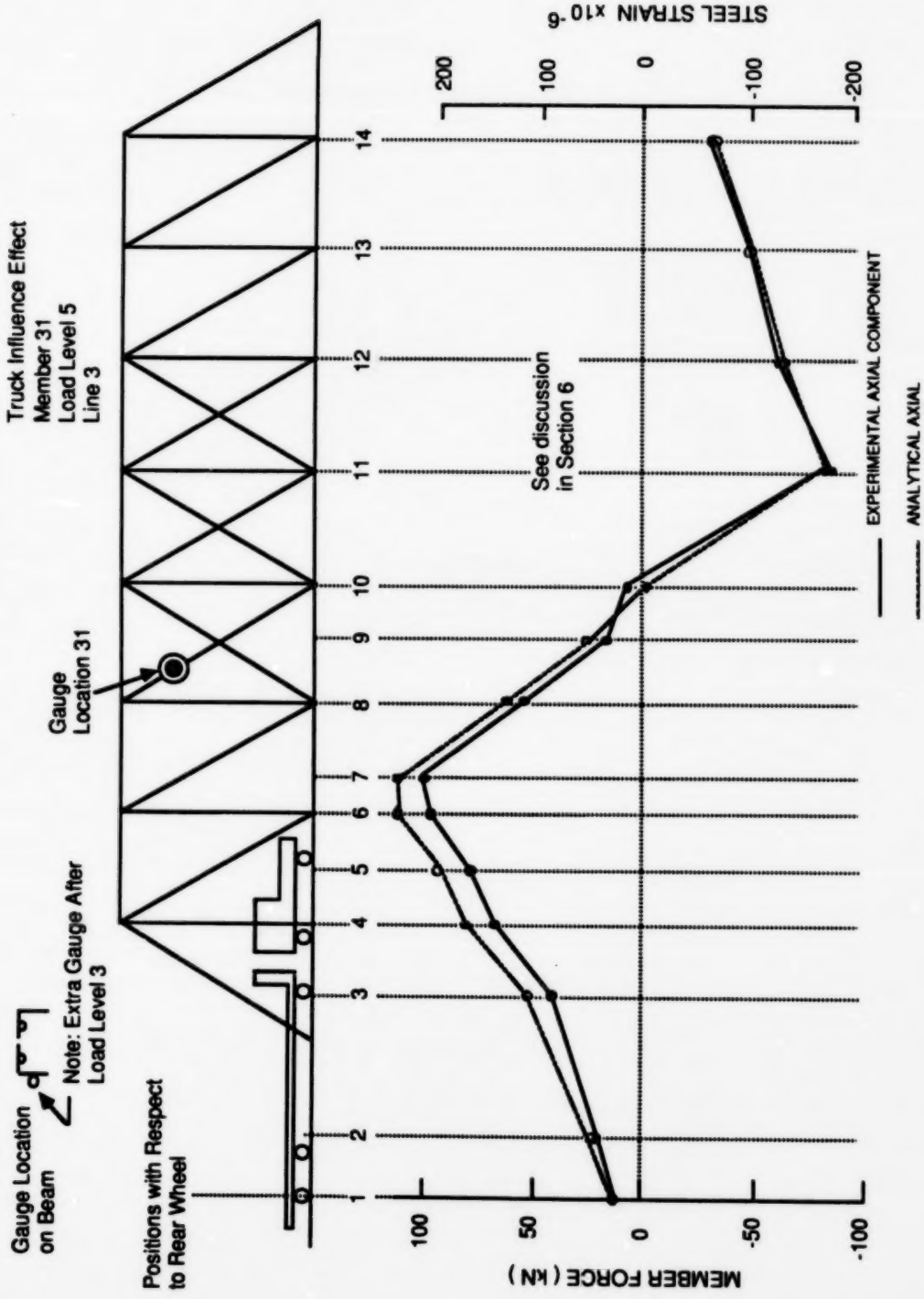


Figure 75/ Member Forces and Strains–Steel T· Span No. 2, Member 31, Load Level 5

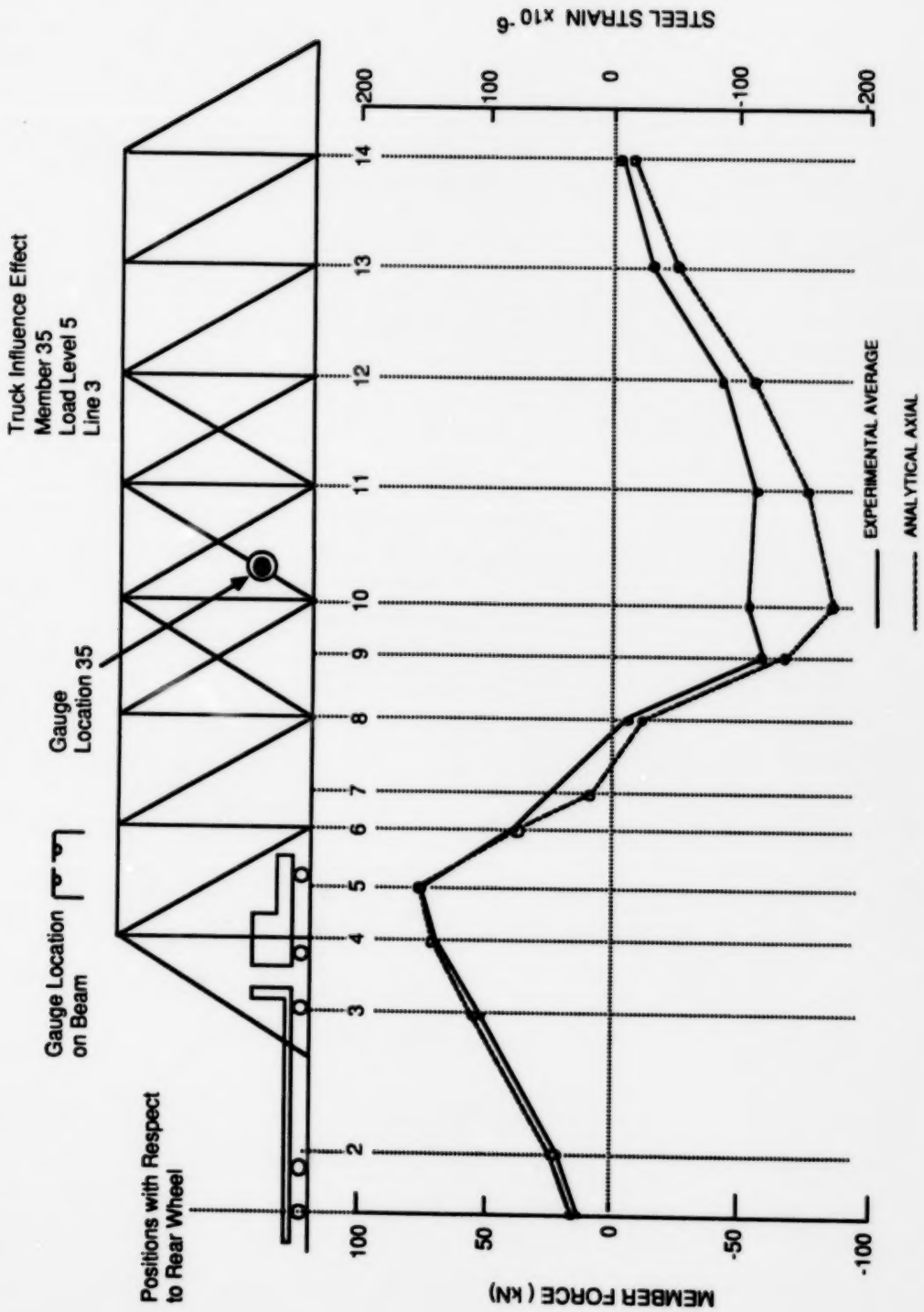


Figure 76/ Member Forces and Strains–Steel Truss, Span No. 2, Member 35, Load Level 5